MECHANISMS AND REMEDIATION OF CUT BATTER FAILURES ALONG THE QUEENS ROAD AT WAINIGASAU AND NAMUKA I LAU, VITI LEVU, FIJI ISLANDS.

A THESIS SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR

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BY

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ABSTRACT

The first cut batter failures along the Queens Road, Viti Levu occurred in 1979 following a prolonged high precipitation event. This has continued to occur through to 2009 and this thesis research is centred on understanding the mechanisms of recurring cut batter failures occurring at Namuka i lau and Wainigasau.

The Namuka i lau and Wainigasau cut batters are cut into highly weathered polymict conglomerates belonging to the Veisari Sandstone. Each site comprises three benches at approximately 9 m in height. The Namuka i lau benches are 1 m wide with 50ºslope faces and the Wainigasau benches are 3 m wide with 60ºslope faces. Field investigations revealed previous landslide activity at the Namuka i lau site in contrast to the Wainigasau site which showed a lack of evidence of previous landslide activity.

XRD analysis determined the clast and matrix material to be composed of quartz, hematite and kaolinite with quartz compositions ranging from 65% - 100%, hematite compositions from 1% - 25% and kaolinite composition percentages ranging between 1% and 25%. Other laboratory studies determined the material to have a high water content (Namuka i lau – 47.2% and Wainigasau – 44.9%), low unit weight (Namuka i lau – 14.5 kN/m^3 and Wainigasau – 15.8 kN/m^3), low permeability (K for Namuka i lau – 2.78 x 10^{-7}m/s and Wainigasau – 6.66 x 10^{-7}m/s), high plasticity, low cohesion (c_r = 3.9 kPa) and a residual friction angle of 15.0 º.

Factors of safety calculated for the cut batters are low (FS = 0.4) due to the low values of cohesion and friction angles used in the modelling process. Sensitivity analysis determined that the highest factors of safety were achieved when both the cohesion value was increased to 20 kPa and the friction angle increased to 30º using the Bishop Simplified Method of slope stability analysis for Wainigasau (FS = 1.28) and Namuka i lau (FS = 1.23).

ACKNOWLEDGEMENTS

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The Queens Road in Viti Levu, Fiji, is the main highway and access for motorists for Southern Viti Levu. It is the convenient access between the main centres of Suva, Navua, Sigatoka and Nadi.

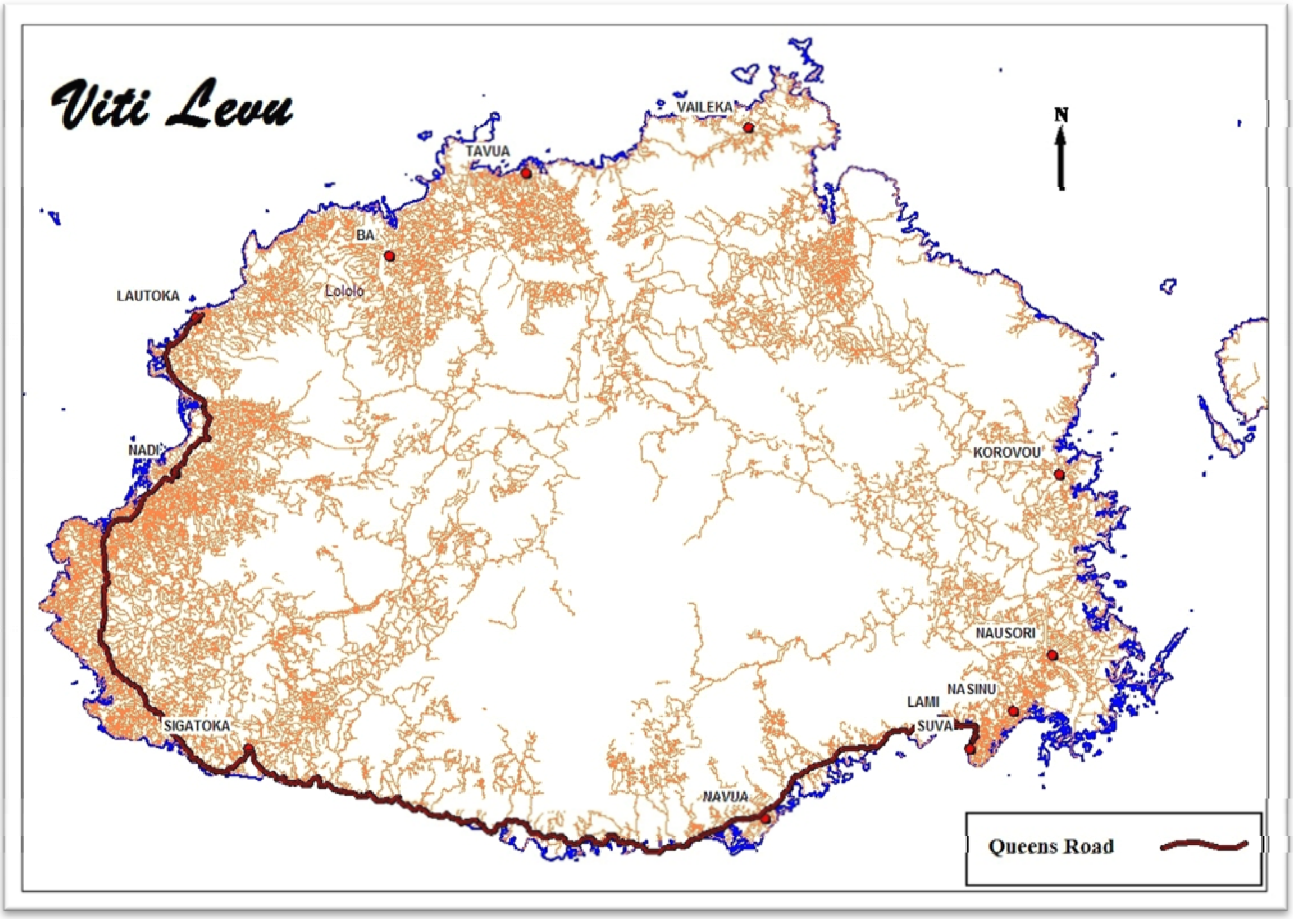


Figure 1.1 Map of Viti Levu road network showing extent of Queens Road

The Queens road was constructed in the early 1960's by the UK based company Dillingham Wilkinson Green after winning the government tender under the Public Works Department.

The cut batters along the Queens road are excavated in intensely weathered Veisari Sandstone formation. This Miocene sedimentary unit, has 2 members: a sandstone which includes fine to pebbly sandstones and a polymict volcanic conglomerate and the second member which is a volcanic conglomerate which is principally derived from the Mau andesite.

Since the completion of construction of the Queens Road, there have been numerous rotational and translational slides along the Queens highway with 3 main extreme events being recorded in 1979, 1980 and 1986 where extensive damage to roads and housing was recorded. A portion of the Queens road affected during the 1980 event required repositioning (Greenbaum et al, 1995).

This research project focuses in on the two areas of Wainigasau and Namuka i lau where the cut batters which were initially constructed during the construction of the Queens Road have become unstable and recent events in 1996, 2003, 2007 and 2008 have seen the failure of cut batters at these sites. The proposed research intends to initially map all current failures in the project area and then investigate the mechanisms of failure in the cut batters. The research will also carry out a modelling exercise of the cut batters to analyse their current slope stabilities and in this process determine the factors of safety for the cut batters and investigate methods to which the cut batters may be stabilised.

The specific hypothesis that this research is based on are listed as follows:

- That cut batter failure is due to a combination of deep weathering and complex hydrogeological conditions weakening the exposed rock mass
- That changed hydrogeological conditions due to clay formation and fracture closures/infilling creates conditions conducive to deep seated rotational movements in highly weathered rocks that have been over steepened.
- That understanding clay mineral types and weathering history will identify key triggers for batter instability and permit identification of possible remediation measures

- Development of an engineering geological and geotechnical model is critical to the formulation of mitigation strategies which are to be part of the project.

1.2 OBJECTIVES

The specific objectives for this research project can be summarised into the following 5 major objectives listed as follows. They are to:

- Prepare corridor maps (engineering geology and geomorphological) showing failure sites, types and relevant cross sections
- Develop an engineering geology and hydrogeological model for the cut batters that have failed
- Collect representative samples for clay mineralogy and other soil property tests
- Conduct stability analysis incorporating all relevant field and laboratory data to permit identification of remedial options and
- Recommend mitigation measures including but not limited to surface flows, seepage in bedrock and stable batter designs based on properties determined on rock and soil samples collected

1.3 LOCATION OF STUDY

Fiji lies between longitudes 174^{o}east and 178^{o}west of Greenwich and between latitudes 12^{o} and 22^{o} south.in the south west pacific with its nearest neighbouring countries being Vanuatu to the west, Tonga to the southeast and Samoa to the northwest. Fiji is made up of 332 islands, which have a total land mass of 18,333 square kilometres.

The study area is located on Viti Levu, the largest island in the Fijian group of islands, which has a total land mass of 10,429 square kilometres.

The two sites of investigation (Namuka-i-lau and Wainigasau) lie along the Queens Road, which is adjacent to Viti Levu's southern coast on the windward eastern side of Viti Levu. This region has higher rainfall and humidity levels as compared to the western side. The Viti Levu central plateau extends through the centre of Viti Levu from the Nakauvadra ranges of northern coastal town of Rakiraki to the Serua Hills extending up from the southern coast between Navua and Sigatoka.

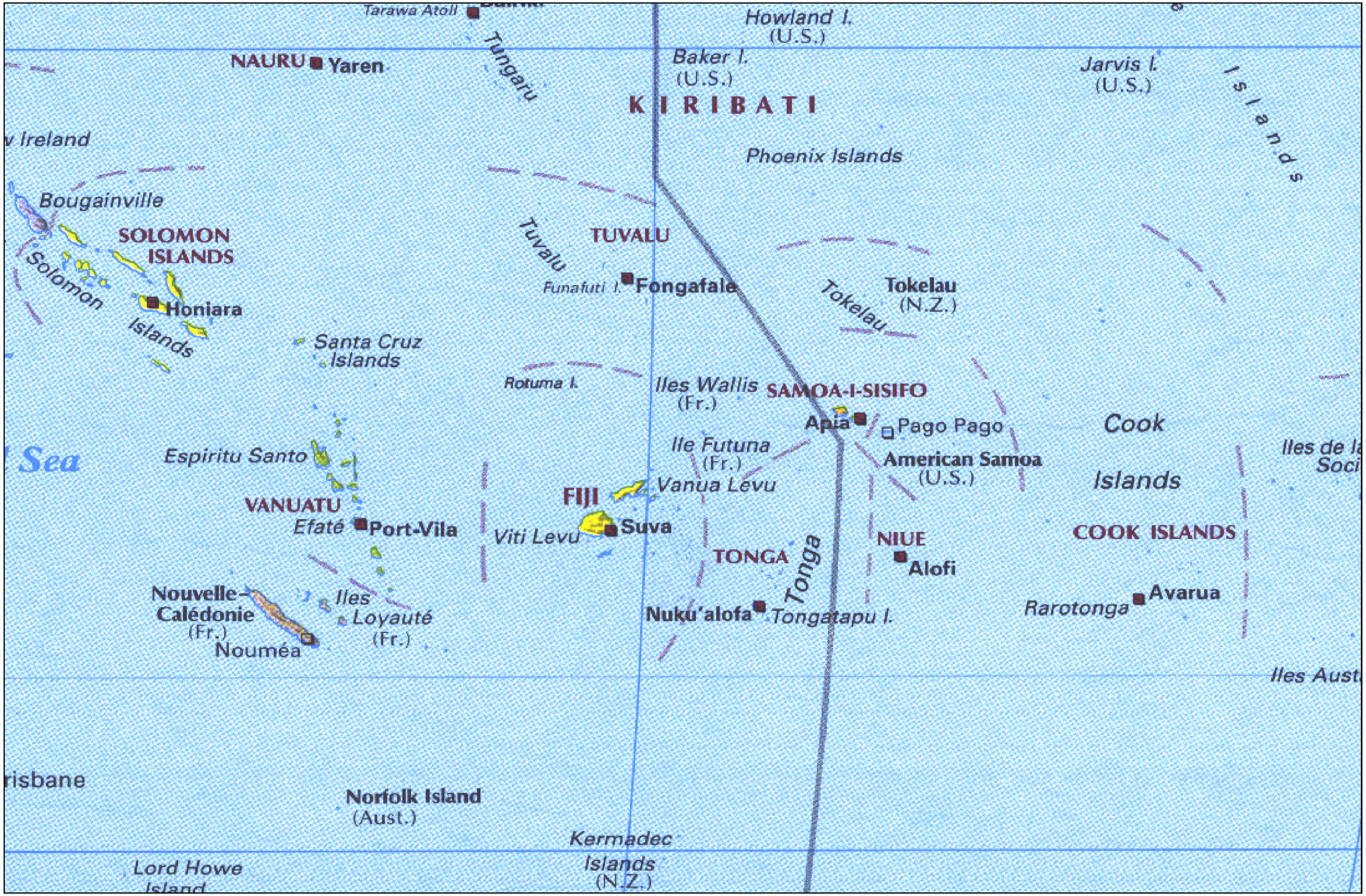


Figure 1.2 Map of the south pacific showing Fijis location in reference to other countries

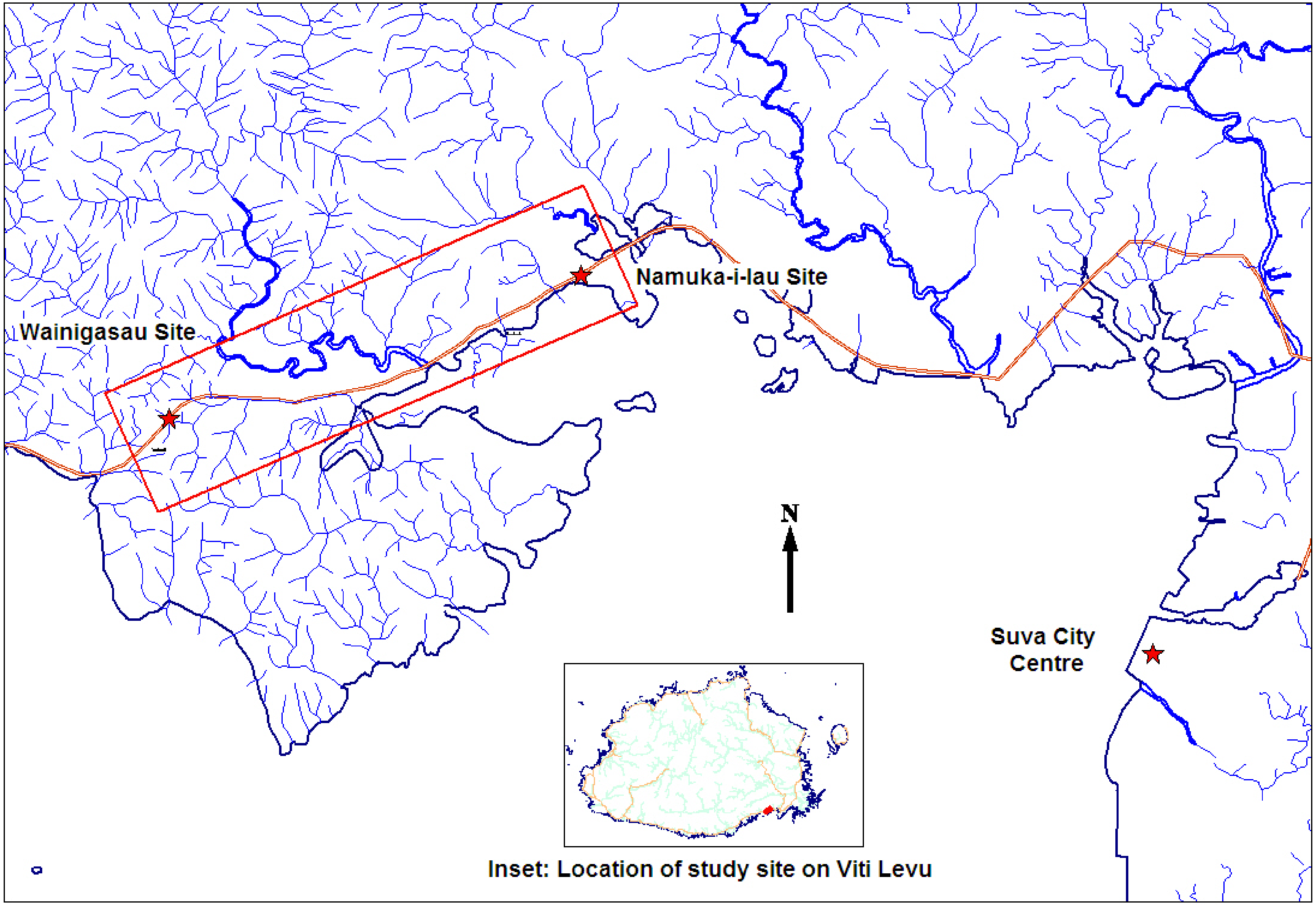


Figure 1.3 Study location area showing the Namuka-i-lau and Wainigasau sites

1.4 CLIMATE

The Fijian archipelago lies within the zone of the Southeast Trades and consequently there is a marked difference in the climate between the lowland areas of the south and east, and those of the north and west (Greenbaum et al, 1995). The average temperatures change only about 2 to 4°C between the coolest months (July and August) and the warmest months (January and February). Around the coast, the average night-time temperatures can be as low as 18 to 20°C and the average day-time temperatures can be as high as 30 to 32°C. Southeastern coastal areas and the high interior often experience persistent cloudy humid weather.

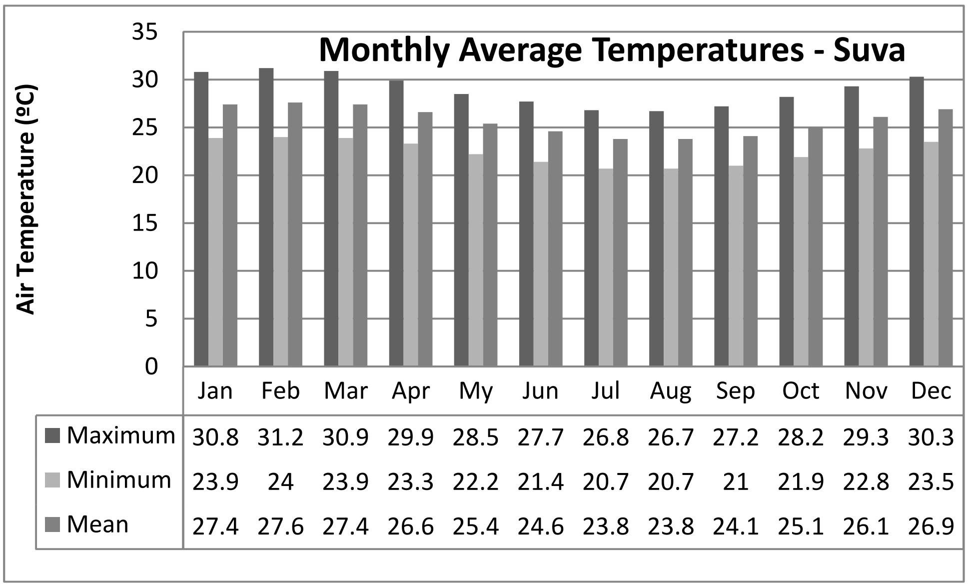


Figure 1.4 Monthly average temperatures for Suva

Rainfall is highly variable and mainly orographic in that it is influenced by the island topography and the prevailing southeast trades. The southeast trade winds are saturated with moisture and any high land mass lying in their path receives much of the precipitation. The mountains of Viti Levu and Vanua Levu create wet climatic zones on their windward sides, and dry climatic zones on their leeward sides, hence the main islands have pronounced dry and wet zones.

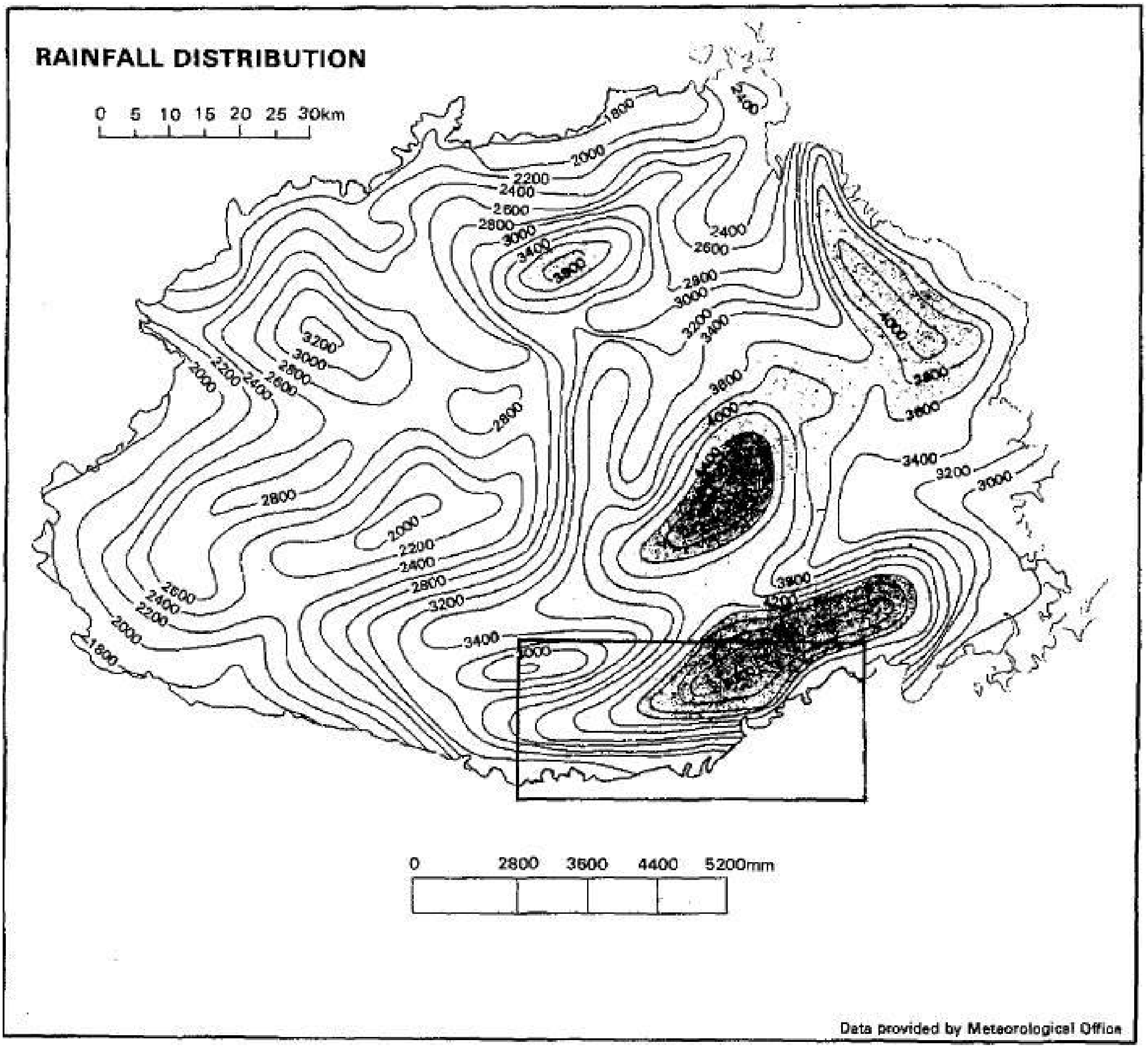


Figure 1.5 Rainfall Distribution map for Viti Levu (Greenbaum at al, 1995)

Fiji experiences a distinct wet season (November to April) and a dry season, controlled largely by the north and south movements of the South Pacific Convergence Zone, the main rainfall producing system for the region. Much of the Fiji's rain however falls in brief heavy showers.

Rainfall is usually abundant during the wet season (November to April), especially over the larger islands, and it is often deficient during the rest of the year, particularly in the 'dry zone' on the north-western sides of the main islands. In the drier half year, from May to October, the heaviest rainfall occurs on the windward (south-east) parts of the larger islands. Annual rainfall in the dry zones averages around 2000mm, whereas in the wet zones, it ranges from 3000mm around the coast to 6000mm on the mountainous sites. The smaller islands receive amounts intermediate between those on the wet and dry sides of the larger islands.

The south-eastern parts of main islands generally receive monthly total rainfall of 150mm during the dry season and 400mm during the wettest months. These parts of the islands have rain on about six out of ten days for the dry season and about eight out of ten days in wet season. The north-western parts of these islands are in the rain shadow and receive generally less than 100mm per month during the dry period. The variation in the monthly totals between the two zones during the wet season is little. The wettest month is usually March and the driest month is almost always July. During the wet season, brief heavy afternoon showers and thunderstorms are common (Meteorological Climate Data, 2000).

The study areas of Namuka i lau and Wainigasau lie between the main centres of Suva and Navua where the average annual rainfall is 3023 millimetres for Suva and 3573 millimetres for Navua based on data collected between the years 1971 and 2000.

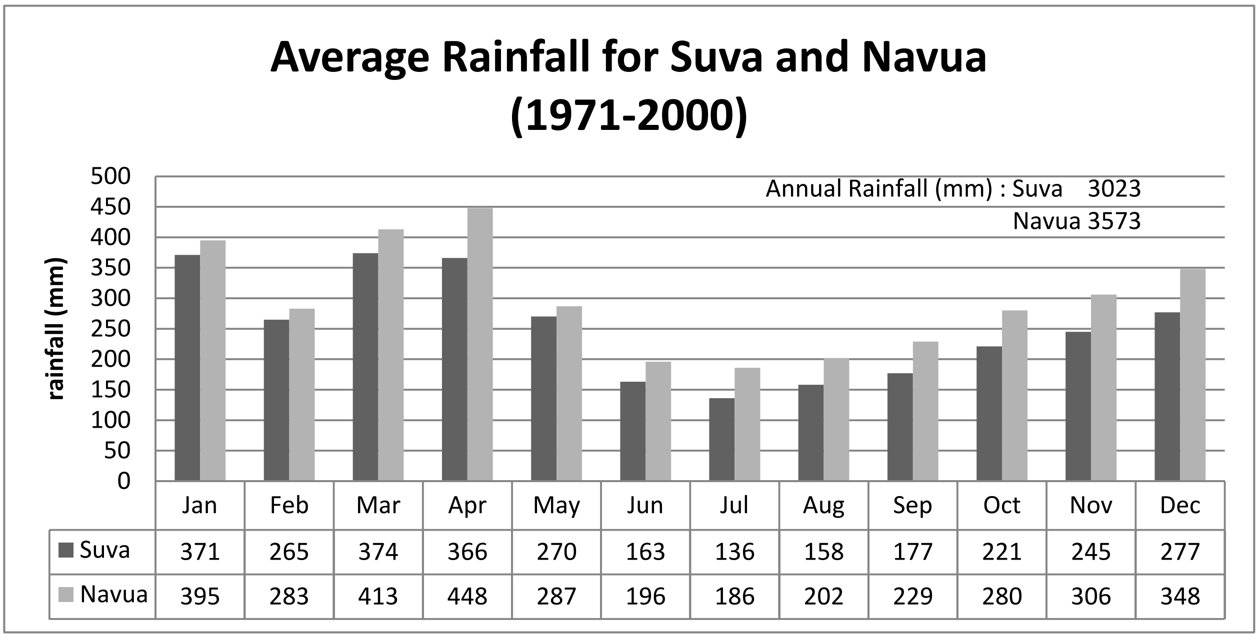


Table 1.1 Average rainfall for Suva and Navua.

1.5 REGIONAL GEOLOGY

1.5.1 Regional Setting

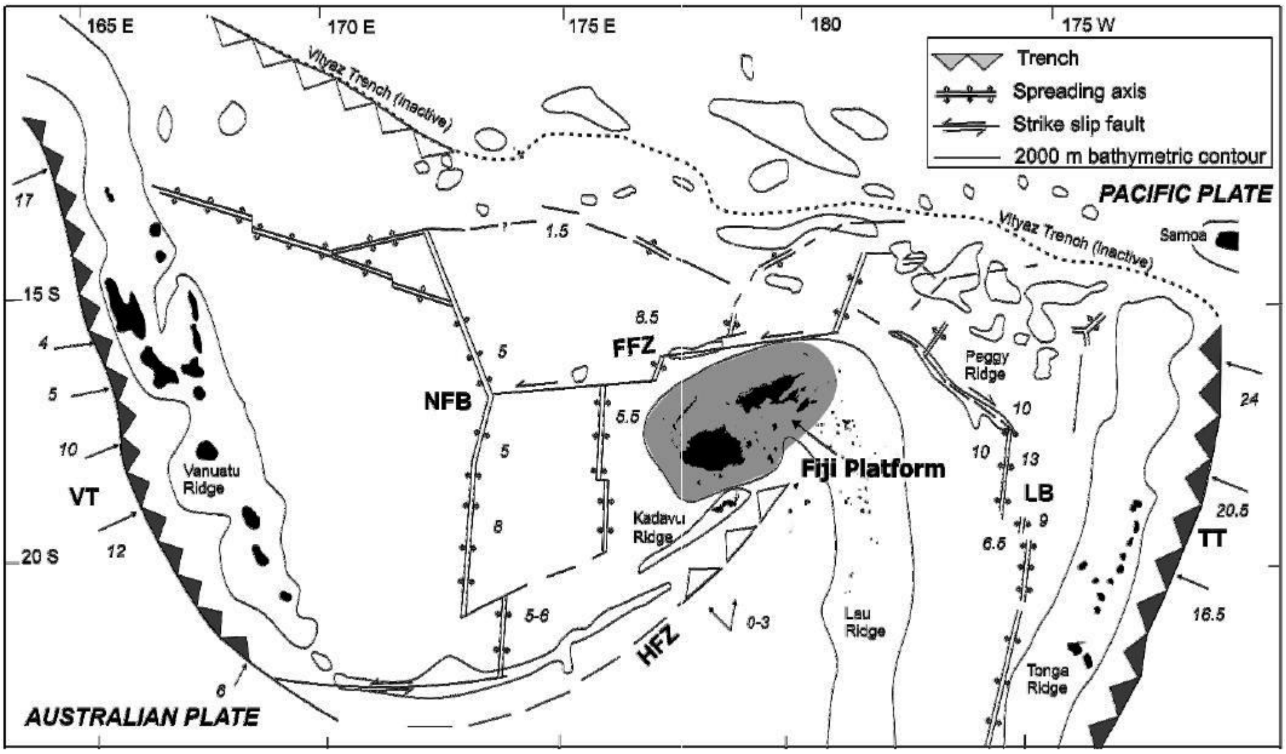


Figure 1.6 Tectonic setting of the Fiji region showing major structural units (Rahiman, 2006)

The two main islands of Fiji, Viti Levu and Vanua Levu, rise from a shallow shelf, the Viti Levu and Vanua Levu platforms. The main basin that seperates them is covered by the Bligh Waters which are just over 1000 metres in depth. The two main islands of Viti Levu and Vanua Levu therefore are regarded as one major structural unit as the geologic structure between them is probably continuous. The other major structural unit is the Lau ridge on which are located the islands and reefs of Lau Group, which are situated on the eastern side of Fiji archipelago. East of this are the Tonga ridge and Tonga Trench (TT). The Kadavu Group, the Moala Group and various other islands, reefs and seamounts, rise from water more than 2000 metres deep whilst several other volcanic edifices encroach on the Fiji Platform and are separated from it by water depths in the range of 1000 metres to 2000 metres. Fiji includes one island and an island group outside the archipelago, the Rotuma group of islands being located approximately 630 kilometres north of Suva. Several basins surround the Fiji archipelago. To the west and east, the Fiji platform and Lau ridge are bordered by passive margins against the North Fiji Basin (NFB) and Lau Basin (LB) respectively. The Fiji platform is bordered to the north by a transform structure named the Fiji Fracture Zone (FFZ). The south of the Fiji platform, the Kadavu Ridge and a few smaller islands are separated from the South Fiji Basin by another transform structure referred to as the Hunter Fracture Zone (HFZ). The Koro basin is a re-entrant of deep water within the Fiji archipelago; its crust of uncertain origin but thought to be Oligocene in age and part of the South Fiji Basin (Rodda, 1994).

1.5.2 Tectonic Setting

The landmasses of Fiji are products of coalesced arc related volcanic, plutonic and volcaniclastic sedimentary rocks from a series of volcanic island arcs spanning the Cenozoic era. The geology is predominantly mapped as repeated sequences of volcanic lithostratigraphic units portraying lateral facies variations from coarse to fine volcaniclastics centred around remnant eruptive centres which have been characterised geomorphically or in more eroded terrain by the presence of volcanic stocks and sub-volcanic plutons. The rocks have undergone episodic periods of deformation attributable to, and controlled by regional tectonic processes. Tectonism has also, in part, controlled development of large intervolcanic sedimentary basins. Periods of emergence recorded in the stratigraphy are attributable to tectonic uplift or eustatic sea level changes or both. The magmatic and petrologic history of Fiji is reflective of primitive, mature, rift and post-subduction related episodes of Fiji’s tectonic history (Rahiman, 2006).

The entire southwest pacific region extending from the Solomon islands to New Zealand is dominated by plate convergence between the Pacific and the Indo-Australian plates. The present loci of convergence are the Solomon, New Hebrides, Tonga, Kermadec, and New Zealand

(Hikurangi) trenches. The Tonga – Fiji - New Hebrides segment of the plate boundary referred to as the interarc region is anomalous in several respects: (1) the convergent boundary is discontinuous jumping from westward subduction at northern Tonga to eastward subduction at the southern New Hebrides (2) the interarc region is one portion of the convergent plate boundary that is not dominated by plate subduction (3) the broad geologically complex Fiji platform sits within the plate boundary and (4) two east trending bathymetric deeps, the Vitiaz Trench and the Hunter Fracture Zone mark the sites of former subduction zones (Hamburger and Isacks, 1987 in Begg and Gray, 2002).

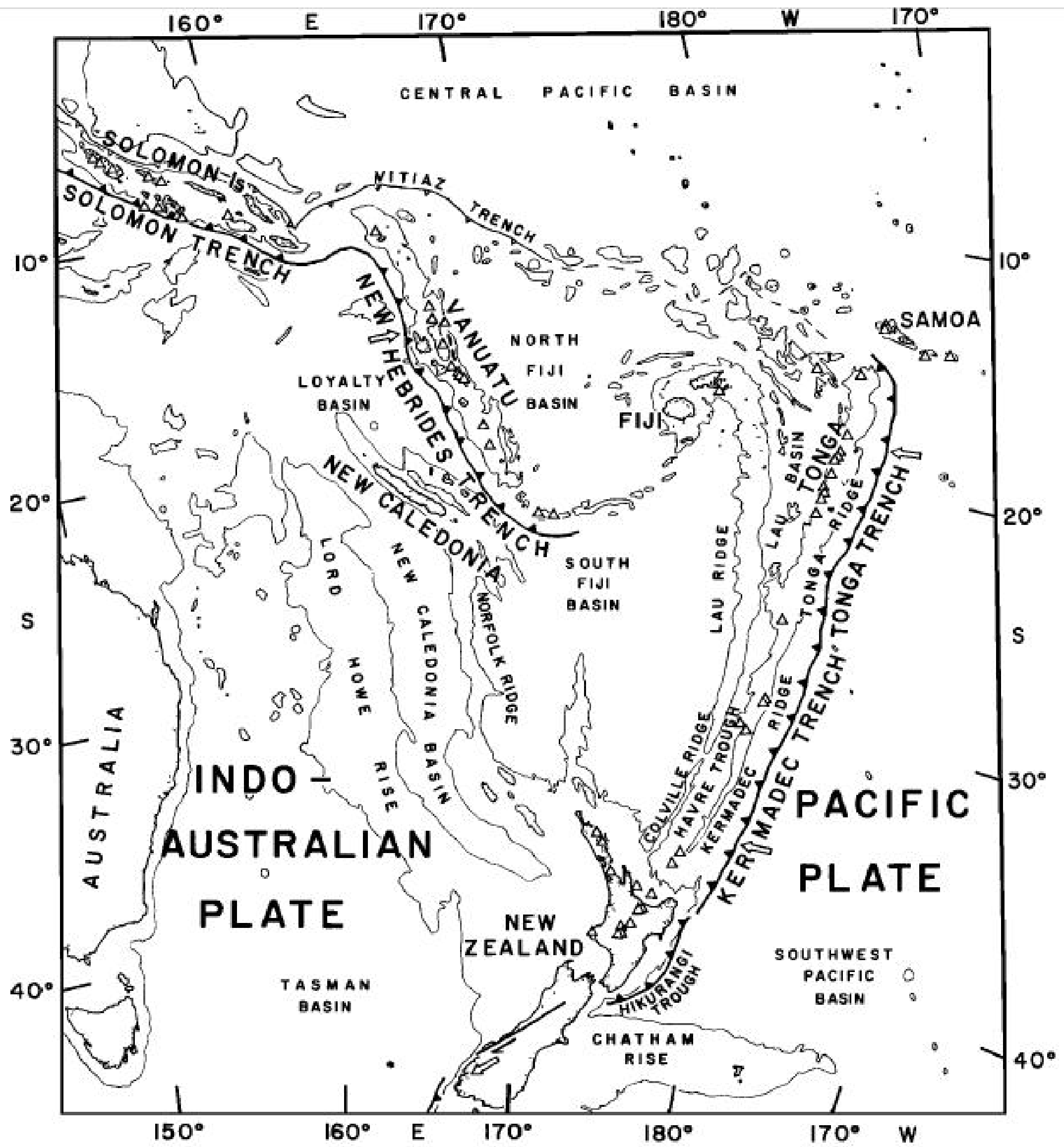


Figure 1.7 Tectonic and morphological features of the Pacific/Indo Australian plate boundary (Hamburger, 1987)

The Fiji Islands are situated on a prominent offset of the convergent boundary between the Pacific and Australian tectonic plates (Figure 1.7). This offset marks a broad zone of diffuse spreading and transform faulting (Hamburger and Isacks, 1988 in Begg and Gray, 2002) accommodating divergence of the east facing Tonga arc-trench system and the west facing Vanuatu arc trench system by left lateral wrenching and opening of the North Fiji Basin. Active subduction is along the present-day Vanuatu and Tonga-Kermadec trenches. The low level of earthquake activity south of Viti Levu and the dominant strike-slip fault plane solutions indicate that subduction along the Hunter Fracture Zone (Figure 1.8) has possibly ceased (Hamburger and Everingham, 1986 in Begg and Gray, 2002).

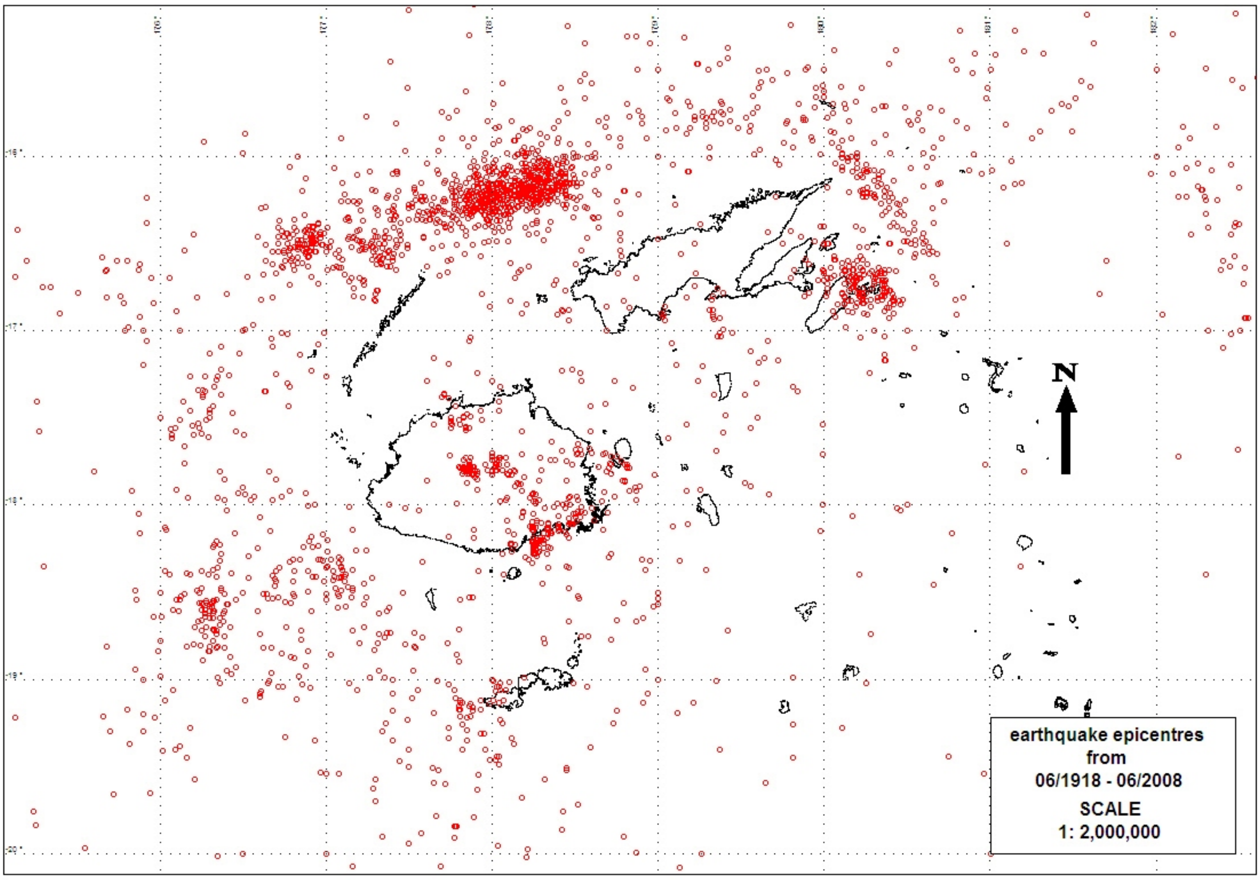


Figure 1.8 Earthquake epicentres from 1918-2008 showing low level earthquake activity south of Viti Levu as compared to high level of earthquake activity associated with the Fiji Fracture Zone north of Vanua Levu

This means that the Fiji Platform is now part of the Australian plate (Rodda, 1993 in Begg and Gray, 2002). The boundary between the Pacific and Australian plates is regarded to be the seismically active Fiji Fracture Zone (Hamburger and Everingham, 1986 in Begg and Gray, 2002). Recent paleomagnetic data suggest that rotation of the Fiji platform stopped abruptly at 3 Ma with the development of a well-defined spreading centre in the North Fiji Basin (Taylor et al., 2000 in Begg and Gray, 2002). Current seismicity data show that while NE-SW shortening dominates in the vicinity of the left lateral Fiji Fracture Zone, it is progressively overshadowed by N-S shortening farther south on the Fiji Platform (Hamburger and Isacks, 1988 in Begg and Gray, 2002). The component of NE-ENE shortening may reflect transfer of stresses associated with sinistral slip of the Fiji Fracture Zone. In the Lau Basin and eastern North Fiji Basin current stress fields indicate N-S shortening and E-W extension along NW and NE striking strike-slip faults (Hamburger et al., 1988 in Begg and Gray, 2002).

Pliocene shoshonitic and calc-alkaline volcanic centres dominantly occur along three lineaments in the Fiji region. Seven shoshonitic and high K calc-alkaline volcanic centres define the ENE trending Viti Levu lineament (Rodda, 1993 in Begg and Gray, 2002). The largest of these is the shoshonitic Tavua Volcano on the island of Viti Levu (‘‘T’’ in Figure 1.9c). This lineament extends from northwest Viti Levu through Vanua Levu and coincides with the NE alignment of volcanic centres on the island of Tavueni. Two other lineaments, the NNW trending Lomaiviti lineament immediately east of Viti Levu and the ENE trending Vatulele-Beqa lineament immediately south of Viti Levu (Gill and Whelan, 1989; Rodda, 1993 in Begg and Gray, 2002), are represented by island chains on margins of the Fiji Platform and are dominated by Pliocene shoshonitic plus lesser calc-alkaline volcanism.

Further south of Viti Levu, just off the southern margin of the Fiji Platform, the Kadavu Islands lie immediately north of the Hunter Fracture Zone and trend between NE and ENE. They are composed of basaltic to dacitic shoshonitic to high-K and medium-K calc-alkaline volcanics ranging in age from 3.2 Ma to <1 Ma (Rodda, 1993 in Begg and Gray, 2002). There is one small volcano of alkali-basalt.

To the west of Viti Levu, the Yasawa groups of islands are dominated by late Miocene basaltic and andestic volcanism. Post-Miocene folding about axes parallel to the NNE to NE strike of the island chain is accompanied by southeast over northwest thrusting (Rodda, 1993 in Begg and Gray, 2002). East of Viti Levu, the islands of the Koro Sea consist of basaltic volcanoes with diverse geochemistry, including tholeiitic, calc-alkaline, shoshonitic, and alkaline (Rodda, 1993 in Begg and Gray, 2002).

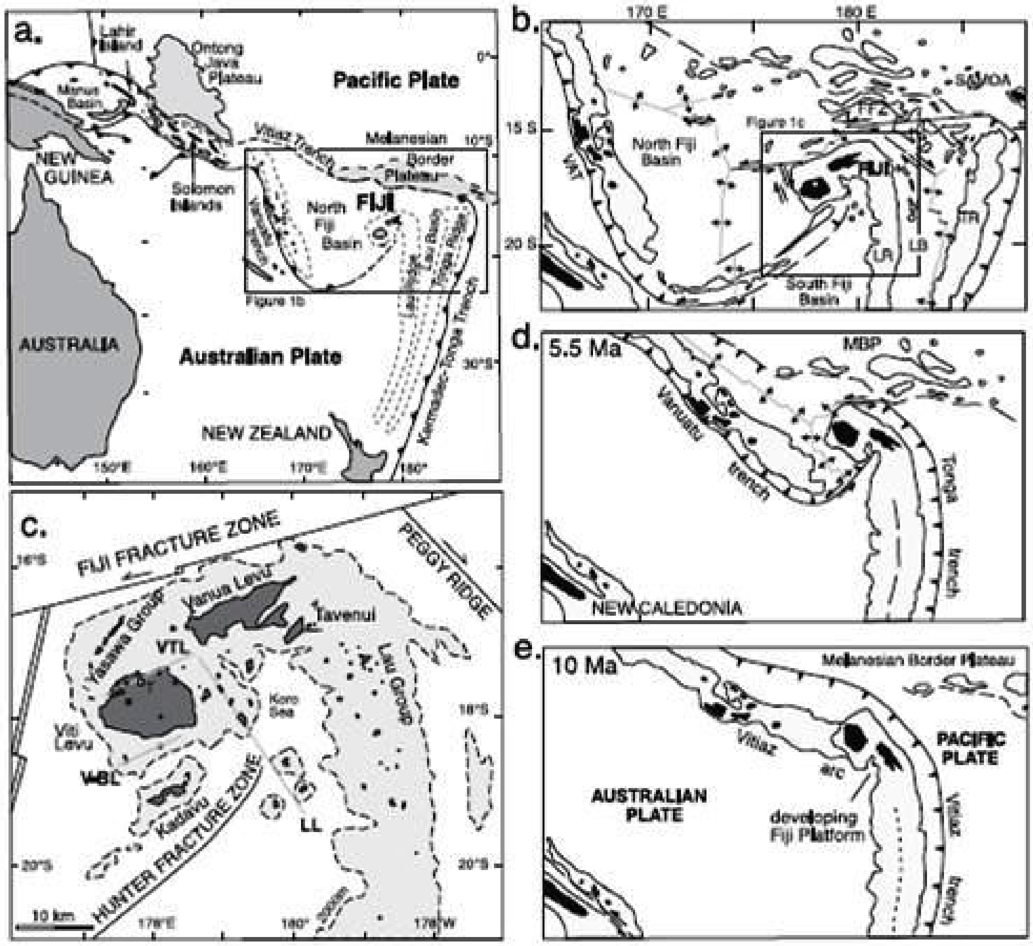


Figure 1.9. Tectonic setting (Figures 1.9a–1.9c) and tectonic reconstructions (Figures 1.9d and 1.9e) of the Outer Melanesian region (a) Map of the Fiji platform and north end of the Lau Ridge showing the major islands in the Fiji area, the major early Pliocene volcanoes of Viti Levu, the major seafloor fracture zones, and part of the spreading center of the Fiji Basin; Shoshonitic volcanoes, including the Tavua Volcano (T), are shown by squares and calc-alkaline volcanoes by circles. (b) Tectonic features of the northeastern segment of the plate boundary between the Australian and Pacific plates showing the Outer Melanesian Arc of the southwest Pacific, trenches and ridge systems, and oceanic plateaus (adapted from Kroenke [1984]). Fiji, as part of the Fiji Platform, consists of a series of islands at the north end of the Lau Ridge, with the North Fiji Basin formed as part of a spreading center. (c) Present plate configuration. (d) Reconstruction at 5.5 Ma. (e) Reconstruction at 10 Ma. In Figures 1.9a–1.9e the Australian plate is fixed and the east-west convergence rate between plates was assumed to be 9–10 cm yr^{-1}. Shading represents submarine depths <2000 m. Abbreviations are as follows: VT, Vitiaz trench; VAT, Vanuatu trench; LR, Lau Ridge; LB, Lau Basin; TR, Tonga Ridge; FFZ, Fiji Fracture Zone; LL, Lomaiviti lineament; V-BL, Vatulele-Beqa lineament. Long dashes denote southern margin of the Melanesian Border Plateau (MBP). The open square (Figures 1.9b and 1.9c) denotes the location of the Tavua Volcano (Gill and Whelan, 1989).

1.6 GEOLOGY OF SOUTHEAST VITI LEVU

The oldest basement rocks in southeast Viti Levu belong to the Wainimala Group and the Savura Volcanic Group. They are weakly metamorphosed (greenschist facies) lavas and volcaniclastic sedimentary rocks. The Wainimala rocks are intruded by tonalite and gabbro stocks of the Colo Plutonic Suite which are Middle to Late Miocene in age. The Tuva group disconformably overlie the Wainimala rocks and are considered to be the detritus of the Wainimala and Colo rocks. The Tuva Group are said to be of Middle to Late Miocene in age (see Figure 1.10).

Volcanic and sedimentary rocks of the Medrausucu Goup overlie the Wainimala and Colo rocks with a marked unconformity, or are partially downfaulted against them in the east and likewise to the west by the Navosa Sedimentary Group. The Medrausucu and Navosa Sedimentary groups are said to be of Late Miocene to Late Pliocene in age. The Medrausucu Group consist of volcanic rocks derived from the Namosi, Mau, Nakobalevu and Nasinu volcanoes and also contemporaneous and younger marine sediments. These sedimentary rocks occur mostly in the down-faulted Sovi, Navua and Rewa basins amidst the older basement rocks. The islands of Yanuca and Beqa consist of Pliocene volcanic rocks and the predominantly limestone island of Vatulele also has occurrences of volcanic rocks of similar age. The Koroimavua and Ba volcanic groups overlie all other rocks to the north and consist of Late Miocene to Pliocene calc-alkaline and shoshonitic volcanic rocks.

The Verata Sedimentary Group disconformably overlies strata of the Medrausucu Group in the Rewa Basin and other areas in the southeast. The Verata Sedimentary Group rocks are Late Pliocene and possibly early Pleistocene shallow shelf strata. Pleistocene fluvial gravels also unconformable overlie the Verata Sedimentary Group rocks and older rocks along the western margin of the Rewa Delta and Holocene alluvial and deltaic deposits occur in the Navua and Rewa river deltas. The coastal lowlands have several metres of Holocene inter-tidal deposits underlying alluvial cover deposits (Rahiman, 2006).

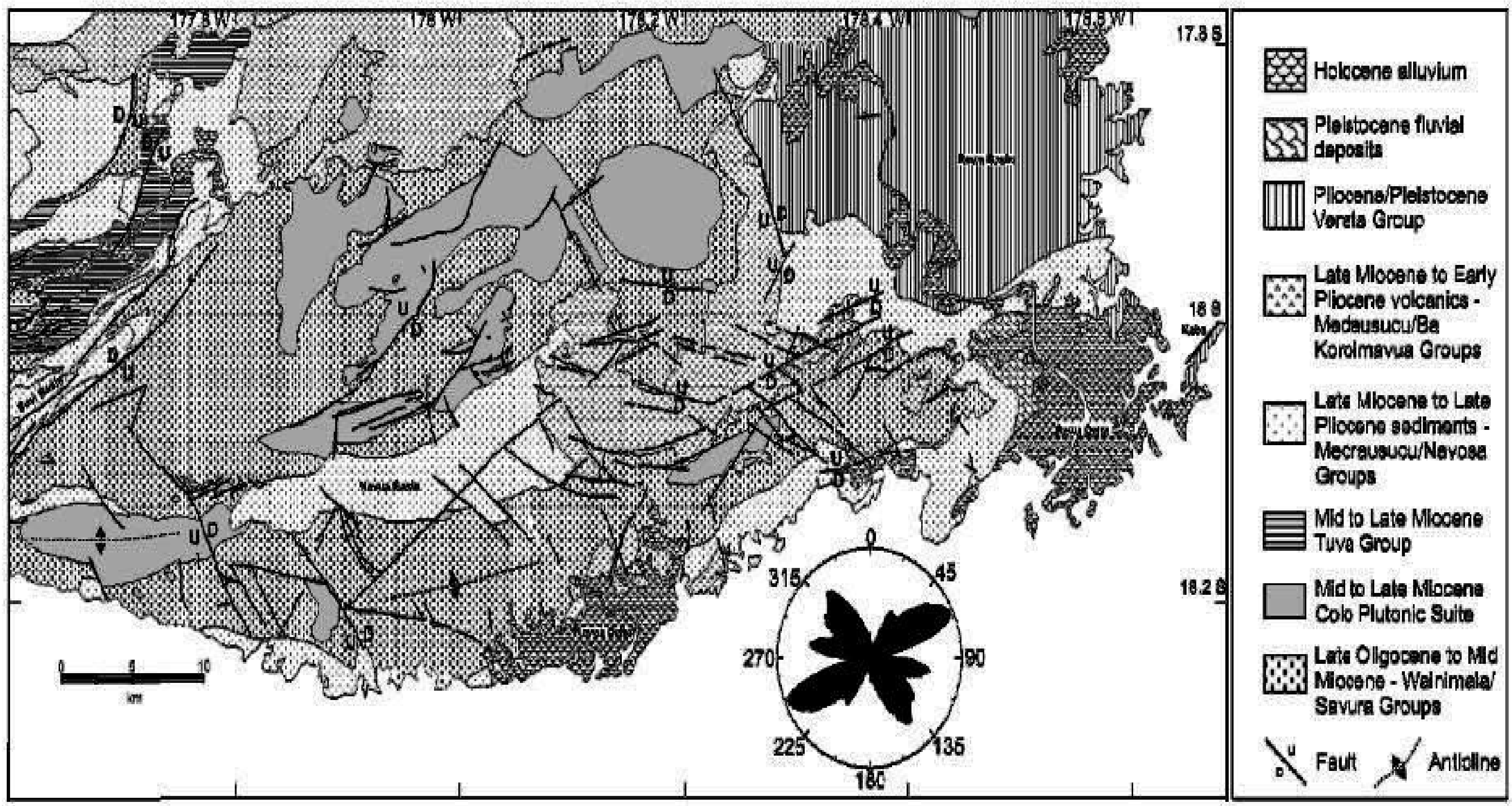


Figure 1.10 Geology of southeast Viti Levu: Rose diagram shows orientation of mapped faults (Rahiman, 2006)

The study area is dominated by the Upper Tertiary Medrausucu Group (Miocene-Pliocene). The group consists of (a) andesitic epiclastic rocks with minor flows known as the Namosi Andesites and (b) penecontemporaneous marine sediments namely the Navua mudstones, Waidina and Veisari Sandstones, Suva Marls and Serua Conglomerates.

The Namuka-i-lau and Wainigasau sites both lie within the Veisari sandstones. The Veisari sandstones have a continuous outcrop for 17 miles forming a strip of lowland about 2 miles wide along the southeastern coast of Viti Levu from Mau to Veisari. The Veisari sandstone passes laterally westwards into the Serua conglomerates and eastwards into the Suva Marls. The Veisari sandstone has two subdivisions: (a) a sandstone member which includes sandstones, pebbly sandstones and conglomerates and (b) a volcanic conglomerate which is derived from a Mau andesite plug (Band, 1968).

The Veisari sandstones are made up predominantly of andesitic material derived from the Namosi andesites and some material from the andesitic Mau volcano. The volcanic conglomerates which originate from the Mau volcanic centre contain sub-rounded boulders of augite andesite and hornblende andesite and overlie bedded sandstones. Hornblende andesite volcanic breccias are interbedded with sandstones in this area generally contain abundant hornblende crystals, indicating that volcanic activity and sedimentation were contemporaneous (Band, 1968).

1.6.1 Medrausucu Andesitic Group

The formation characterising the Medrausucu Andesitic group is the Namosi andesite occurring in the upper Waidina region and represented by pillow lava, normal flows, volcanic breccias and conglomerate and tuff. The early volcanic rocks are black or brown augite andesite containing very calcic plagioclase in a glassy groundmass. Analyses suggest however that the rocks have an overall andesitic composition (Woolnough, 1903). Higher in the sequence, pale grey to white hornblende andesite is dominant. From paleontological evidence relating to other formations of the Group, these volcanic rocks are inferred to be upper Miocene or lower Pliocene.

The volcanic rocks diminish in importance laterally and upward and give way to various formations of sandstone and mudstone, the rocks furthest from the andesite being marl with some limestone. Fossil faunas have been given a wide range of ages, but upper Miocene and lower Pliocene (i.e. tertiary g) seem most likely (Ibbotson, 1960). Several formations within the group show basal conglomerates which were partly derived from tonalite, or which unconformably overlie tonalites of the Colo Plutonic Suite (Rodda, 1967).

1.6.2 Alluvium

Alluvial deposits shown on the map are mostly modern deltas, that of the Rewa river being the largest. Deposition of sediments in the south east of the island has probably been almost continuous since at least the beginning of the Pliocene, producing part of the Medrausucu group, the Verata group and the Rewa delta. Some alluvial deposits are very thick, a drill hole at the head of the Navua delta penetrated 185 ft of gravel without reaching bedrock. The Sigatoka river has a negligible delta, the alluvium shown there being mostly sand dunes (see Figure 1.11).

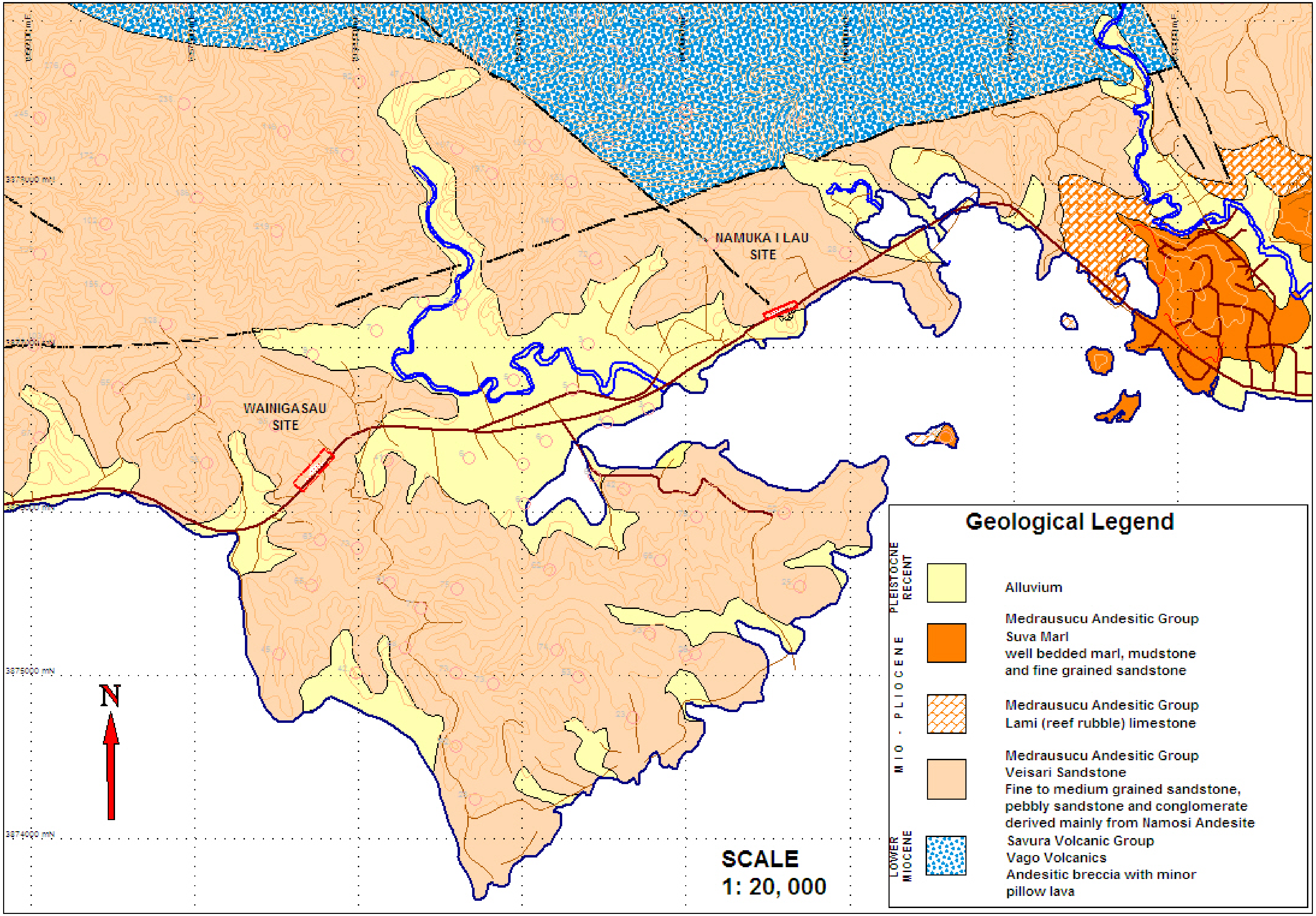


Figure 1.11. Geology of the study area

1.7 SOIL TAXONOMIC UNIT DESCRIPTION

Soils in Fiji have been extensively documented and classified by Twyford and Wright (1965) who produced a comprehensive report and map covering Fiji. The major soil groupings are sub-divided into a series of range numbers. Range numbers correspond to various geographical and geological conditions. These range numbers also incorporate slope class symbols varying from flat to vey steep.

In their classification, Twyford and Wright (1965) have suggested that the two major groups of soil forming materials in Viti Levu are tuffaceous sedimentary rocks and basic to intermediate volcanics. Soils derived from the intermediate volcanic rocks and their associated sedimentary derivatives are generally reddish brown to red as a result of oxidation of iron oxides. They may also appear black where the sediments are calcareous or where the volcanics are enriched with pyroxene phenocrysts. Basaltic rocks give rise to reddish brown, brown and black soils which vary in appearance according to age, stability and depth of weathering. Generally but with a few exceptions, the depth of a soil profile correlates with the age of the parent rock. The colluvial, alluvial and aeolian deposits prevalent on foothill and floodplain areas are derived from a mixture of rock types but mineral grains from the intermediate andesitic types are dominant. Where siliceous rocks are dominant in the area, the resulting soils derived from these rocks are usually rich in quartz grains.

The dominant soils in south east Viti Levu are classified as humic latosols derived mainly from rocks of basic to intermediate compositions. In general terms, these latosols are the product of leaching in a very strong tropical weathering environment. They are essentially clays in composition but behave like clay loams or loams in the field.

The following are descriptions of two major soil classes of the upland zones and the description of the soil class covering majority of the river deltas and river valleys in the study area.

The soils derived from rocks of andesitic and basaltic composition predominant in the hilly country of the Navua and Serua Hills. There are probably synonymous with the Serua and Lobau steepland clay and boulder clay unit. The soil profile on moderate slopes typically shows a sequence as follows:

Reddish brown friable clay with a fine to very fine nutty structure.	7 cm
Red friable clay with a fine angular fragmental structure	108 cm

On very steep slopes, the pedological soil profile is usually more than 40 centimetres in thickness. These soils are nutrient rich and support the natural forest and preferred for subsistence farming. The local variant of this type of soil is the Batiwai Clay which are quite thicker sometimes extending to 6 metres but are located on more gentle slopes to rolling hills.

The Naraiyawa steepland soils is similar to the soil grouping derived from rocks of acidic composition and shows a typical profile as follows:

Dark brown friable stony silty clay loam with a weak blocky structure	10 cm
Light brown, friable, very stony, sandy clay with a coarse blocky structure to a fine, blocky and coarse granular structured sandy soil containing abundant weathered rock	24 cm

The deltas and river valleys of the Suva and Namuka harbours are covered mainly by the Tokotoko clays which are a gley soil which are related to the nigrescent soils. The Tokotoko clay is defined by Twyford and Wright (1965) as a soil with a mottled topsoil indicating fluctuating water tables, overlying a gleyed subsoil which implies permanent water saturation; a drained phase of the Tokotoko clay (45b); Tokotoko clay on sandy clay (45c), the lower horizon of an estuarine origin. Their parent rocks are of intermediate and basic composition and parent material, recent weakly weathered alluvial rocks. The Tokotoko clays are very poorly drained and rarely have a water table depth more than 50 centimetres. The permeability is very low but increases drastically where drained, structural cracks develop.

A typical profile description is as follows:

moist, dark greyish brown silty clay loam with	
weakly developed, coarse, blocky structure	17 cm
mosit, olive grey to dark greyish brown clay loam	
weakly developed coarse prismatic structure	20 cm
moist, , firm, strong brown to yellowish brown clay,	32 cm
wet, light olive grey to pale olive clay, massive and firm	65 cm

A study carried out by Lawson (1993) showed that geotechnical tests carried out on steepland latosolic soils of south east Viti Levu revealed that while there seemed to be few marked textural or compositional differences in the soils, there were pronounced differences seen in their engineering properties. This highlights the difficulty of attempting to relate pedological classifications on a regional soil map to engineering properties of the materials and resulting susceptibility to landslides.

1.7.1 Weathering Variations in soil profiles

Differences in the properties of soil result from the gradational effects of weathering, both vertically and laterally in the profile. A typical weathering profile comprise a mature intensely weathered residual soil grading to successively less altered rock at depth. This gives rise to differences in material strength properties within the vertical profile. The intensity of tropical weathering process alters the mechanical and physical properties of the residual soils bears little or no resemblance to the parent rocks from which they were derived. Another point to note is that weathering increases and is more intense at or around fissures in the parent rocks so there are lateral variations as well as vertical variations in the chemical and physical properties of components of any soil profile. Lateral variations in properties on steep residual soil slopes may also result from surface material which has attained near equilibrium with the physic-chemical environment, being continuously removed by landslides and erosion. This exposes less weathered or 'immature' parts of the weathered profile to weathering processes. This leads to soils of markedly different material and engineering properties to be juxtaposed against each other on the steepland slopes of south east Viti Levu (Twyford and Wright, 1965).

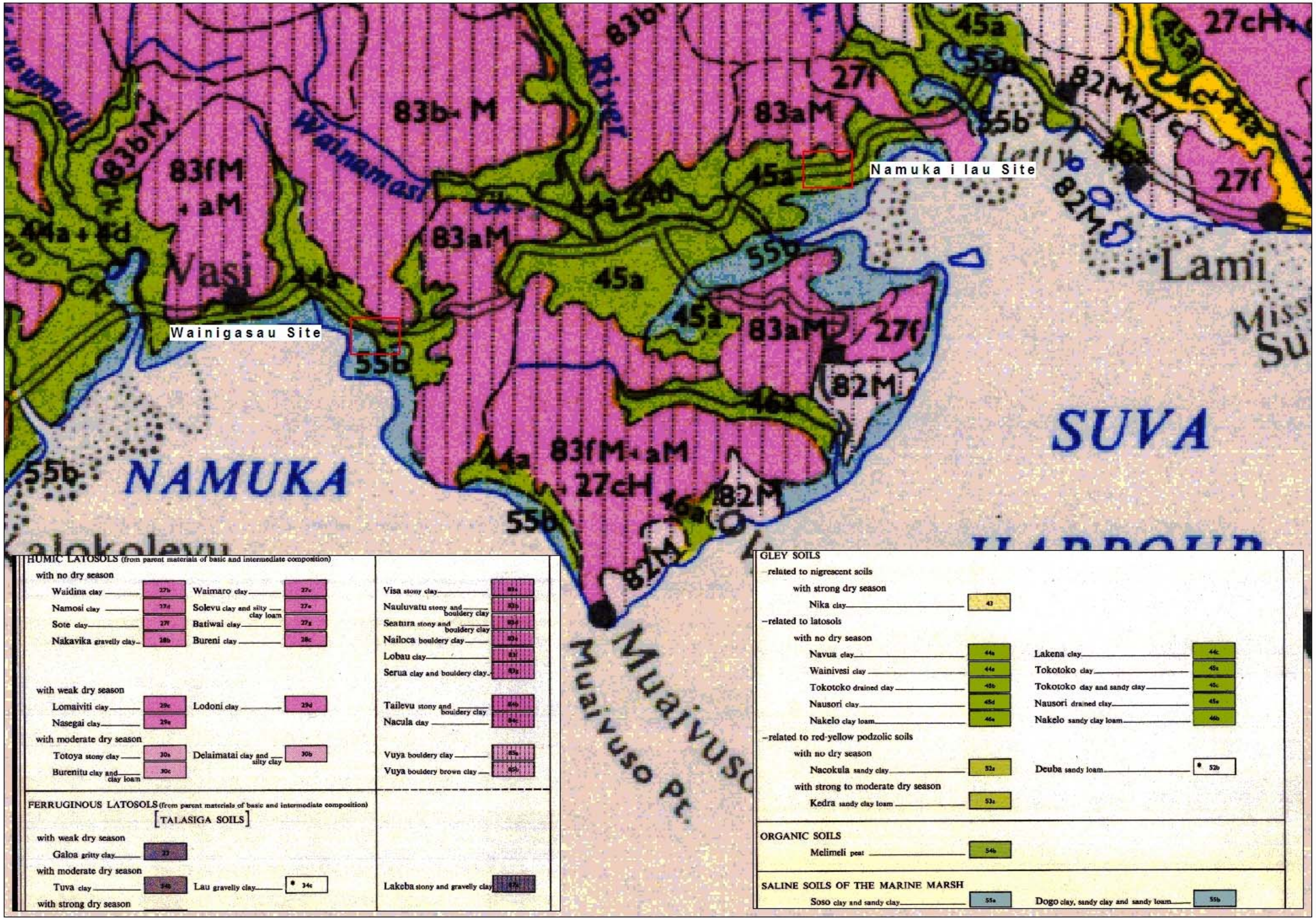


Figure 1.12 Soil classification of the Wainigasau and Namuka i lau areas (Twyford and Wright, 1965)

1.8 THESIS METHODOLOGY

To thoroughly investigate the mechanisms for the cut batter failures at Wainigasau and Namuka-i-lau, a variety of desk studies had to be carried first to lay the foundation for the planning of the physical investigations and laboratory investigations that would be carried out on the study areas.

1.8.1 Literature Review

The desk study of the study areas included a literature review of past investigations that had been done on the surrounding areas which included, but were not limited, to geological mapping, geotechnical investigations, landslide mapping and materials testing of soil and rock samples.

1.8.2 Aerial Photo Interpretation

Aerial photo interpretation is a powerful method in producing landslide inventories, which show the spatial distribution of mass movements with different characteristics. It enables the operator to identify objects simultaneously over large areas for either a direct assessment of widespread landslide occurrence following a distinct trigger event (e.g. rainfall, earthquake) and/or a characterization of the general spatial landslide distribution for a given area. The latter aim of investigation is often used to identify landslides according to their magnitude in terms of spatial extend and approximated age of occurrence. After respective characterization, their distributions and clustering and further spatial relationships to other factors such as slope aspect, vegetation cover, etc. can be developed and might enhance the understanding of landslide distributions in the past. It is a very cost effective method, which requires a mirror stereoscope and aerial pictures of different scales, and ideally also for different dates (Melzner et al, 2007).

The aerial photographs used were provided from the Fiji Land Information Services at a scale of 1:50, 000 and 1:15, 000. A multi-temporal approach was used to estimate the timing of occurrences of landslides and construction of cut batters within the study area. Aerial photographs from the years, 1978, 1982, 1985, 1986, 1994 and 1998 were collated but varying degrees of coverage were available for the study area for the years collected.

An aerial photo interpretation was carried out for both the Wainigasau and Namuka-i-lau areas. Multiple sets of aerial photographs from the above mentioned years were interpreted using stereo pairs and a binocular stereoscope.

Objectives of the aerial photo interpretations were to identify:

1. Past failure types including flows, falls, slides, topples and spreads.
2. Main characteristics of past failures such as head scarps, failure deposit, tension cracks, boundaries of deposit, etc.
3. Other structural features such as faults, major ranges, cliffs and other lineaments
4. Main hydrologic features such as rivers, creeks, ponds, etc
5. Major outcrops and excavations.

A simple and appropriate legend was also constructed for the aerial photo interpretations with appropriate symbols for each of the features identified.

1.8.3 Geological Mapping

The two major failure areas of Namuka-i-lau and Wainigasau were mapped in detail to determine geological and hydrogeological conditions of the areas. Geomorphological mapping of the area identified past failures and debris from historic events in the area. Structural mapping was carried on the sites to identify presence of jointing and fracture patterns present in the cut batters and also to identify any occurrences of faulting.

Samples from the cut batters were collected and sent to University of Canterbury for engineering testing. Composition analysis of the rocks was determined through X-ray diffraction methods. Engineering testing of the clays also included determination of unit weights, Atterburg limits, grading analysis and the production of grading curves. Ring Shear tests were also carried out on the clays to determine shear strength properties. Permeabilities of the clays were determined by testing of the clays using the falling head test.

Historical information from Department of Roads, Department of Mineral Resources and the Meteorological Office were collated to assist with the proposed research into causative factors of the failure of the cut batters and also mitigation strategies for the stabilisation of the cut batters.

In determining the mechanisms of failure of the cut batters, mitigative strategies were proposed for the stabilisation of the cut batters based on the results determined through this study. These included mitigative strategies for surface flow, seepage in bedrock and cut batter slope angles based on soil and rock engineering properties.

1.8.4 Auguring

Auguring was carried out at the Wainigasau site to determine the depth of the weathered andesitic breccias and also determine the depth of the water table. Auguring was terminated at six metres because of equipment difficulties.

1.8.5 Permeability Testing

Permeability testing was carried out on samples from both the Wainigasau and Namuka-i-lau site. The samples tested for permeability were undisturbed samples which were sampled insitu by driving the double ring permeability cell into the weathered breccias. The undisturbed samples were then sealed in plastic before transport back to the laboratory for testing. Testing was carried out using the WF26010 Falling Head Permeability Cell.

1.8.6 XRD Analysis

Representative samples for clasts and matrix were collected from the weathered breccias for XRD analysis. The clasts and matrix samples were also sealed and heat sealed in plastic before being sent to the University of Canterbury. This was done to minimise drying of the samples. Some clay components change compositions as they dehydrate.

X-ray crystallography is the most common method for determining the structure of molecules. It involves a simple process of placing pure crystals under an intense beam of x-rays of a single wavelength to produce a series of reflections or spots, whose intensities are measured for every orientation of the crystal. This is achieved by rotating the sample and measuring each reflection intensity for each crystal orientation analysed. All this data is then combined computationally with prior information about molecular structure to produce an atomic resolution model for the sample.

X- ray Diffraction or Powder diffraction is a variance of the above method where x-ray diffraction is used on powder samples. Ideally, every possible crystalline orientation is represented equally in the powder sample, leading to smooth diffraction rings around the beam axis rather than the discrete spots observed for single crystal diffraction. Each ring corresponds to a particular reciprocal lattice vector in the sample crystal which is identified computationally with prior 'reference' information.

Relative to other methods of analysis, powder diffraction allows for rapid, non-destructive analysis of multi-component mixtures without the need for extensive sample preparation. This allows for quick analysis of unknown materials and performs material characterization in such fields as metallurgy, mineralogy, forensic science, archaeology and the biological and pharmaceutical sciences.

The diffractometer used was the Philips PW1820/1710 X-ray Diffractometer system.

1.8.7 Ring Shear Tests

Soil mechanics essentially studies the effects of soils deforming as a continuum under increasing stress as well as the effects of dilation within the soil. Ring shear was conducted because the samples being tested were from the failed cut batter material. Shear testing was carried out using the ring shear test apparatus to determine the residual strength of our samples and the sample tested was the matrix material of the andesitic conglomerate found at the two study areas. The conglomerate is essentially matrix supported and the sample tested first had to be sieved to remove the coarser component of the conglomerate before testing.

1.8.8 Geotechnical Modelling

Geotechnical modelling was carried out using the SLIDE version 5.0 software to create models of the existing cut batters at Namuka i lau and Wainigasau. These were used to calculate factors of safety for the cut batters. A sensitivity analysis was also carried out to determine the effects of varying the geotechnical parameters cohesion and friction angle, on the Factor of Safety. A back analysis was also carried out to determine the reinforcement forces that are needed to achieve a factor of safety value of 1.5.

1.9 THESIS ORGANIZATION

This thesis is organised into 6 chapters.

Chapter 1 is the introductory chapter introducing and defining the purpose and objectives of the research. It also delineates the study area and its location firstly in the south pacific and specifically its location on Viti Levu, the largest of islands in the Fiji group.

The tectonic setting of Fiji and regional geology is described, concluding with the methodologies used in the research and layout and organisation of the thesis.

Chapter 2 is the second introductory chapter focussed on a Literature Review carried out as part of the research. The literature review is divided into four sections focussing on:

- the classification of Landslides discusses the Varnes Landslide classification scheme and nomenclature and terms used in this research
- Testing Procedures discusses and describes standard testing methods used in this research and calculations used to determine specific properties of the materials tested
- Mitigation options discusses the various options that are used widely in engineering to mitigate similar occurrences of failure as determined in this research and also particular methods that would be best suited to the case studies covered in this research and a
- Synthesis summarising Chapter 2.

Chapter 3 is titled 'Field Investigations' and details the methodologies used and the current geological and geomorphological conditions at the Namuka i lai and Wainigasau sites.

Chapter Four is titled, 'Laboratory Studies' and details the types of laboratory tests and analysis carried out and presents and discusses the results of the analysis.

Chapter Five titled, 'Geotechnical Modelling and Stability Analysis' discusses the methodologies used in the geotechnical modelling process and discusses the results of back and sensitivity analysis carried out. Remedial measures recommended conclude this chapter.

Chapter 6 titled, 'Summary and Conclusions' concludes the thesis report discussing the objectives achieved as a result of this thesis research and concludes with a concluding statement.

1.10 SYNTHESIS

- Viti Levu is the largest island in the Fijian group of islands comprising over 300 islands. The main access along the southern coast is provided by the Queens Road which was constructed in the early 1960's by the UK based company DWG. Since construction, cut batter failures have been occurring along the Queens Road beginning in 1979. This thesis research is centred on studying the mechanisms of failure for cut batters at Namuka i lau and Wainigasau. Specific objectives include the production of relevant geological and geomorphological maps through investigative field studies. To carry out laboratory analysis of the cut batter material to determine properties which will be used in geotechnical modelling processes that will enable us to further understand the mechanisms of cut batter failure and recommend appropriate remedial measures.
- The Namuka i lau and Wainigasau sites are located in southwest Viti Levu which is the windward side of Viti Levu with associated high levels of annual rainfall in excess of 3000 mm.
- The Fiji islands are products of coalesced arc related volcanic, plutonic and volcaniclastic sedimentary rocks from a series of volcanic island arcs spanning the Cenozoic era. The Fiji Islands are situated on a prominent offset of a convergent boundary between the Pacific and Australian tectonic plates and other major structural units for the Fijian group of islands include the Fiji Fracture Zone to the north, the North Fiji basin, Peggy Ridge to the northeast and the Hunter Fracture Zone to the south of Viti Levu. The geology of southeast Viti Levu is dominated by the Miocene-Pliocene Medrausucu Andesitic Group. The Namuka i lau and Wainagasau site geology comprises the Veisari Sandstone which consists of a sandstones, mudstones and andesitic polymict conglomerates. Typical soil

types for the Namuka i lau and Wainagasau areas are humic latosols and gley soils. The Namuka i lau site is classified as Tokotoko clays and the Wainigasau site as Navua clays.

- The research methodologies include a literature review and desktop studies including an aerial photo interpretation. Fieldwork investigations determined the geomorphology and geology of the Namuka i lau and Wainigasau sites. Samples collected during the field investigations were also tested in the laboratory using XRD and grainsize analysis, Atterburg Limits determination, ring shear testing, water content and unit weight calculations. The parameters determined from our laboratory studies were used in our geotechnical modelling analysis to determine factors of safety for the cut batters and to carry out sensitivity and back analyses.

CHAPTER 2

LITERATURE REVIEW

2.1 CLASSIFICATION OF LANDSLIDES

The most widely used and accepted version for the classification of landslides is that presented by Varnes (1978). The classification emphasizes the type of movement and the type of material involved. This is reflected in the naming of the landslides where the first noun describes the material and the second noun describes the type of movement. In the classification, the three material types are rock, debris and earth. There are five main types of movement namely falls, topples, slides, spreads and flows (Cruden and Varnes, 1996). The sixth type of movement proposed by Varnes in 1978, called complex movement, has only been retained by Cruden and Varnes in 1996 as a description of the style of activity of a landslide which involves a combination of two or more principal types of movement.

2.2 NAMING LANDSLIDES

Landslides can be named and classified according to the type of movement and the type of material. In naming landslides, the first noun describes the material and the second describes the type of movement (Cruden and Varnes, 1996). This is summarised in the table below:

Type of Movement	Type of Material		
	Bedrock	Engineering Soils	
		Predominantly Coarse	Predominantly Fine
Fall	Rock Fall	Debris Flow	Earth Fall
Topple	Rock Topple	Debris Topple	Earth Topple
Slide	Rock Slide	Debris Slide	Earth Slide
Spread	Rock Spread	Debris Spread	Earth Spread
Flow	Rock Flow	Debris Flow	Earth Flow

Table 2.1 Abbreviated Classification of Slope Movements (adapted from Cruden and Varnes, 1996)

The naming of landslide can be more elaborate and the suggested sequence provides a progressive narrowing of the focus of the descriptors, first by time and then by spatial location, beginning with a view of the whole landslide continuing with parts of the movement and finally defining the materials involved. Table 2.2 from Cruden and Varnes (1996) summarises the recommended sequence describing activity (including state, distribution and style) followed by descriptions of all movements (including rate, water content and type).

Activity		
State	**Distribution**	**Style**
Active	Advancing	Complex
Reactivated	Retrogressive	Composite
Suspended	Widening	Multiple
Inactive	Enlarging	Successive
Dormant	Confined	Single
Abandoned	Diminishing	
Stabilised	Moving	
Relict		

Description of First Movement			
Rate	**Water Content**	**Material**	**Type**
Extremely Rapid	Dry	Rock	Fall
Very Rapid	Moist	Soil	Topple
Rapid	Wet	Earth	Slide
Moderate	Very Wet	Debris	Spread
Slow			Flow
Very Slow			
Extremely Slow			
Description of Second Movement			
Rate	**Water Content**	**Material**	**Type**
Extremely Rapid	Dry	Rock	Fall
Very Rapid	Moist	Soil	Topple
Rapid	Wet	Earth	Slide
Moderate	Very Wet	Debris	Spread
Slow			Flow
Very Slow			
Extremely Slow			

Note: Subsequent movements may be described by repeating the above descriptors as many times as necessary.

Table 2.2 Glossary for naming of landslide (adapted from Cruden and Varnes, 1996)

2.3 LANDSLIDE ACTIVITY

In the investigation and study of any landslide it is essential to understand the activity of the landslide in terms of its historical, present and future movements and also an understanding of the distribution and styles of movements associated with the landslide.

In describing landslide activity, the following three headings are widely accepted and used:

1. State of Activity which describes what is known about the timing of movements. Active landslide denotes those that are currently moving. They include first time movements and

reactivations. Inactive slides are those that last moved more than one annual cycle of seasons ago and reactivated landslides are those that become active after being inactive. Landslides that have moved within the last annual cycle of seasons but that are not moving at present were described by Varnes (1978) as suspended. Inactive landslides can further be subdivided into dormant and abandoned dependent if the causes of movement remain apparent. If remedial measures have stopped the movement of the landslide then it can be described as stabilised. Relict landslides are those that have clearly developed under different geomorphic or climatic conditions.

2. Distribution of Activity which describes where the landslide is moving. An advancing landslide can be defined as when the surface of rupture is extending in the direction of movement. If the surface of rupture is extending in the opposite direction to that of movement then it is called a retrogressive landslide. If the surface of rupture is extending at one or both lateral margins, the landslide is said to be widening. The term enlarging is sometimes used when the surface of rupture is seen to be extending in one or more directions and diminishing for an active landslide is where the volume of material being displaced is decreasing with time.

3. Style of Activity which describes the manner in which different movements contribute to the landslide. A complex slide was first proposed by Varnes (1978) for those with at least two types of movement but it is now widely accepted that the term complex be limited to cases in which the various movements occur in sequence. The term composite is used to describe landslides in which different types of movement occur in different areas of the displaced mass, sometimes simultaneously. A multiple landslide shows repeated movements of the same type, often following enlargement of the surface rupture and a successive movement is identical in type to an earlier movement but in contrast to a multiple movement does not

share displaced material or a surface rupture with it. Single landslides consist of a single movement of displaced material often as an unbroken block.

2.4 RATE OF MOVEMENT

A scale for the rate of movement for landslides was proposed by Varnes (1978) which was modified by Cruden and Varnes (1996) to convert velocities from feet per second to millimetres per second and also to assign 7 classes increasing in magnitude by multiples of 100.

Velocity Class	Description	Velocity (mm/sec)	Typical Velocity
7	Extremely rapid		
		5×10^3	5 m/sec
6	Very rapid		
		5x101	3 m/min
5	Rapid		
		5x10-1	1.8 m/hr
4	Moderate		
		5x10-3	13 m/month
3	Slow		
		5 x 10-5	1.6 m/year
2	Very Slow		
		5 x 10-7	16 mm/year
1	Extremely Slow		

Table 2.3. Landslide velocity scale (adapted from Cruden and Varnes, 1996)

2.5 WATER CONTENT

The water content of landslide material can be estimated by observation of the displaced material:

1. Dry – no moisture available

2. Moist – contains some water but no free water; the material may behave as a plastic solid but does not flow
3. Wet – contains enough water to behave in part as a liquid, has water flowing from it, or supports significant bodies of standing water
4. Very wet – contains enough water to flow as a liquid under low gradients.

2.6 MATERIAL

The material contained in a landslide may be described as either rock, a hard or firm mass that was intact and in its natural place before the initiation of movement or soil, an aggregate of solid particles, generally of minerals and rocks that was transported or was formed by the weathering of rock. Soil is divided into earth and debris. Earth describes material in which 80 percent or more of the particles are smaller than 2 millimetres. Debris contains a significant proportion of coarse material; 20 to 80 percent of the particles are larger than 2 millimetres, and the remainder are less than 2 millimetres (Cruden and Varnes, 1996).

2.7 TYPE OF MOVEMENT

Kinematics of landslide is one of the principal criteria for classifying landslides. Kinematics of a landslide is defined as 'how movement is distributed though the displaced mass' and understanding the kinematics of a landslide is essential for determining the appropriate response. There are essentially 5 kinematically distinct types of landslide movement: fall, topple, slide, spread and flow.

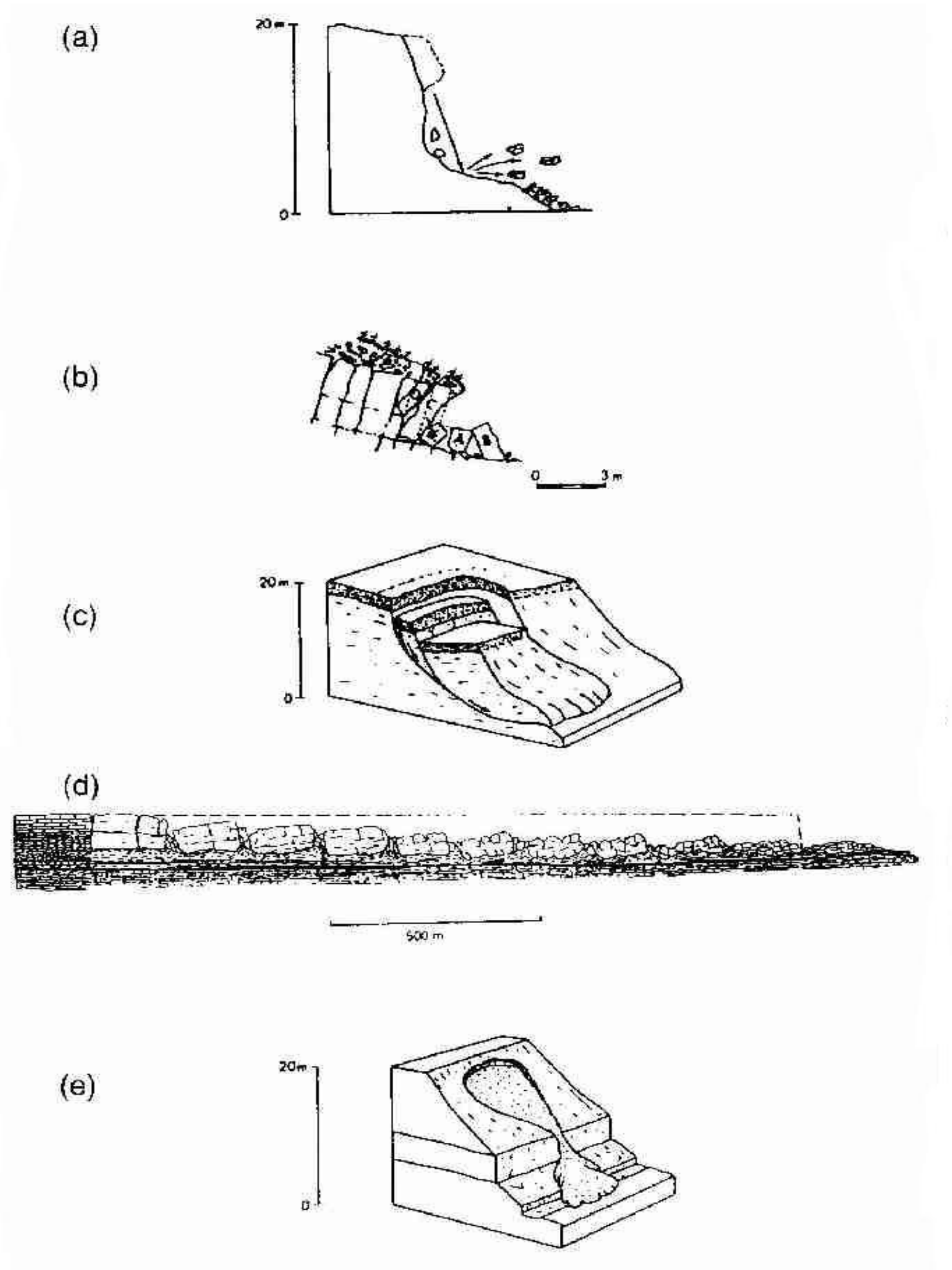


Figure 2.1 Types of landslide movement (a) fall, (b) topple, (c) slide, (d) spread, (e) flow (adapted from Cruden and Varnes, 1996)

2.7.1 Fall

A fall is a detachment of soil or rock from a steep rock slope along a surface on which little or no shear displacement takes place. Movement is rapid to extremely rapid and mode of movement is by falling, bouncing or rolling mainly though air.

Fall failures in Viti Levu

For south east Viti Levu, rock falls comprising blocks up to three metres in diameter have occurred in steep slopes of Suva Marl. Falls may also occur in ‘intact’ residual soils particularly along river banks and coastal cliffs which are being eroded by stream or wave action and from over steepened slopes formed by landslide back scarps and manmade cuts. Examples of these are seen along the Navua River in the upper gorge reaches where the Navua River has incised steeply into the Navua mudstones forming gorges with vertical sides with heights reaching up to a hundred metres. On a regional basis, falls are probably not as significant a hazard as other types of slope failures in south east Viti Levu (Greenbaum et al, 1995).

2.7.2 Topple

Topple is the forward rotation out of the slope of a mass of soil or rock about a point or axis below the centre of gravity of the displaced mass. Topples can be caused by the weight of materials upslope of the displaced mass or by forces generated by water and ice in cracks in the displaced mass. Topples range from extremely slow to extremely rapid sometimes accelerating throughout the movement.

Toppling failure is not common to south east Viti Levu and no events of toppling failure are evident in the literature to date.

2.7.3 Slide

A slide can be defined as a downslope movement of a soil or rock mass occurring dominantly on surfaces of rupture or on relatively think zones of intense shear strain. Movement though does not initially occur simultaneously over what will become the surface of rupture; the volume of displacing material enlarges from an area of local failure.

Sliding failures in southeast Viti Levu

Two main modes of sliding in Viti Levu are:

1. Rotational slides which move along a surface of rupture that is curved and concave upward imparting a backward rotation of tilt to the slipping mass, which sinks at the rear and heaves at the toe.

   In southeast Viti Levu, relatively shallow slumps (generally less than 5 metres in depth) occur mainly in thick, dominantly cohesive (clayey) sequences of residual soil or weathered regolith. Larger, deeper seated slides may involve weathered rock. The backscarps of these slides following initial failure are usually steep, often subvertical, and multiple failure movements are common, resulting in a series of bench-like slip masses and scarps formed as rotational movement retrogresses upslope from the unsupported original backscarp. Because of the concave upward nature of the slip surface in, slumps often reach a state of near stability after only relatively small displacements. This usually results in most of the slumped mass remaining within the area of shear failure. once stability is attained, slumps, in particular, may rapidly degrade to grass or scrub covered cross-slope hummocks or mounds which, if unrecognised, can be readily reactivated if the slope equilibrium is adversely altered by excavation or loading (Greenbaum et al, 1995)

2. Translational slides where the mass displaces along a planar or undulating surface of rupture, sliding out over the original ground surface. Translational slides are generally shallower then rotational slides.

   In southeast Viti Levu, translational slides are most frequent in the residual soils or granular weathered regolith overlying more competent rock, with the depth failure largely controlled by the thickness of weathered debris. Discontinuities remaining as relict features within the weathered regolith also exert an important control on the development and shape of translational slides. Howorth et al. (1993) and Lawson (1993) describe a number of slides in natural and manmade slopes where weathered debris had slid along slickenslide planar surfaces related to pre-existing joint or shear surfaces within the parent rock mass. Translational slides in south east Viti Levu are often triggered when slopes are over steepened by erosion or excavation, as is associated with steep road cuts. Slides in weathered rock debris on steep mountain slopes are frequently triggered by intense rainfalls or earthquake shocks (Greenbaum et al, 1995).

2.7.4 Spread

Spread is defined as an extension of a cohesive soil or rock mass combined with a general subsidence of the fractured mass of cohesive material into softer underlying material. The surface of rupture is not a surface of intense shear. Spreads may results from liquefaction of flow (and extrusion) of the softer material. The rate of movement for spreads is variable and dependent on the mechanism of movement.

Spread failures in southeast Viti Levu

Greenbaum et al (1995) describes superficial creep occurring in environments such as Fiji, where movements arise from soil moisture changes in both fine and coarse grained regolith with movement rates generally less than 10 millimetres per year. Daily temperature changes may also contribute to this form of creep.

2.7.5 Flow

A flow is a spatially continuous movement in which surfaces of shear are short lived, closely spaced, and where the slip surface is usually not preserved. The distribution of velocities in the displacing mass resembles that in a viscous liquid. Earth flows can have some internal deformation with most movement taking place on or immediately adjacent to their boundaries but larger debris flows and debris avalanches will have complete internal deformation (Cruden and Varnes, 1996).

Flow failures in southeast Viti Levu

In southeast Viti Levu, flows involving residual soils and weathered regolith are extremely widespread. Howorth et al (1980) used the term debris flow to describe flows involving regolith in the Wainitubatolu catchment near Korovou village which, in some instances on steep slopes, also show characteristics of debris avalanches. Lawson (1993) states that most slows in the region are debris flows consisting of a wet, variable mixture of material ranging in grain size from clay to boulders.

2.8 CAUSES OF SLOPE FAILURE

Understanding the cause of any landslide is paramount for redesigning and constructing new stable slopes. It is important to be able to anticipate the changes in the properties of the soil within the slope that may occur over time and the various loading and seepage conditions and secondly for the purposes of repairing failed slopes. It is important to understand the essential elements of the situation that lead to its failure so that the best remedial option can be designed for its stabilisation and to ensure that repetition of the failure can be avoided.

Varnes (1978) provided a list of the causes of landslides and Cruden and Varnes (1996) stated that the 3 basic broad types of landslide processes are those that:

1. Increase shear stresses
2. Contribute to low strength, and
3. Reduce material strength.

Water plays a role in many of the processes that reduce strength and is also a component in the many types of loads on slopes that increase shear stresses. It is not surprising that most slope failures involve water contributing to one or more of the causes of the destabilisation.

Another common factor involved in many slope failures is the presence of soils that contain clay minerals. The behaviour of clays in soils is much more complicated in comparison to the relatively chemically inert particles that constitute sands, gravels and nonplastic silts. The mechanical behaviour of clays is determined in part by the physiochemical interaction between clay particles, the water infilling void spaces and ions in the water. Consequently the larger the clay content in the soils, the more active the clay minerals and the greater is its potential for swelling, creep, strain softening and changes in its behaviour.

Duncan and Wright (2005) stated that the fundamental requirement for stability of slopes is that the shear strength of the soil must be greater than the shear stress required for equilibrium. Given this basic condition, the fundamental cause of any slope instability is that by some reason, the shear strength of the soil is less than the shear strength required for equilibrium. This can be reached by either:

1. A decrease in the shear strength of the soil, or
2. Through an increase in the shear stress required for equilibrium.

We will first discuss mechanisms through which shear stresses are increased and then discuss mechanisms through which shear strength is reduced and material strength is reduced.

2.8.1 Increase in Shear Stress

Removal of support

The most common method is through the removal of the toe of a slope and steepening of the slope. This can be achieved through excavation, erosion at the base of the slope by streams, rivers, glaciers, waves and currents. This effectively increases the shear stresses in the soil within the slope, reducing stability.

Imposition of Surcharges

Mechanisms for the imposition of surcharges include:

1. Addition of material or loading of the slope. This increases the shear stress required to maintain equilibrium of the slope.
2. Addition of water through precipitation (rain and snow), flow of surface and groundwater into the displacing mass and even the growth of glaciers. If the water fills cracks at the top of the slope, the hydrostatic water pressure in the cracks loads the soil within the slope, increasing shear stresses and destabilising the slope. Infiltration and seepage into the soil within a slope also increases the weight of the soil, which is appreciable when combined with other effects that accompany increased water content. Other anthropogenic surcharges include construction fills, stockpiles, waste dumps, structural weights, and water leaking from canals, irrigation systems, reservoirs, sewers and septic tanks.

Transitory Stresses

Shear stresses in the slope can be increased through transitory stresses from earthquakes and explosions (both anthropogenic and volcanic). Earthquakes subject slopes to horizontal and vertical accelerations that result in cyclic variations in stresses within the slope, increasing them above their static values for brief periods (fractions of a second). Smaller transitory changes can results from storms, pile driving and the passage of heavy vehicles.

Uplift or tilting

Uplift or tilting or the earth’s surface caused by either tectonic or volcanic forces generally causes steepening of the slopes in the area as drainage responds by increased incision. The cutting of valleys in the uplifted area may cause valley rebound and accompanying fracturing and loosening of valley walls with inward shear along flat lying discontinuities. The fractures and shears may allow the build-up of pore water pressures in the loosened mass and eventually lead to landsliding.

2.8.2 Decrease in Shear Strength

There are several processes that lead to a decrease in shear strength in soils affecting slope stability and clays in particular are most prone to many of these processes.

These processes include:

Increased pore pressure (reduced effective stress)

Rise in groundwater levels and increased seepage in soils is frequently associated with intense prolonged periods of rainfall. Increased water levels in soils may dissolve natural rock cements holding particles together. Increased saturation effectively reduces intergranular pressure and friction and destroys capillary tension. Pore stresses also increase and effective stresses within the slope are decreased. Times for pore pressures increase largely depends on the permeability of the soil. Changes in soils with low permeabilities are slow but may increase rapidly in soils with high secondary permeabilities due to cracks, fissures and lenses of more permeable materials.

Cracking

Dry weather may cause desiccation cracking of weak or weathered rock along pre-existing discontinuities such as bedding planes. Cracks develop as a result of tension in the soils at the ground surface that exceeds the tensile strength of the soil. Once cracks occur in soils, all strength on the plane of the crack is lost.

Swelling (increase in void ratio)

Clays and in particular highly plastic and heavily overconsolidated clays are subject to swell when in contact with water. Low confining pressures and long periods of access to water promote swelling. Skemptons (1964) cases of slides occurring in the overconsolidated London clays where zones of higher water content extended for about an inch on either side of the shear surfaces, indicating that the shear stresses within the developing rupture zone led to localized dilation of the heavily overconsolidated clay.

Development of Slickensides

Slickenslides develop in clays, especially high plasticity clays as a result of shear on distinct planes of slip. As shear displacements occur on a distinct plane, platelike clay particles tend to be realigned parallel to the plane of slip. The resulting smooth surface exhibits a dull lustre where clay readily separates. Slickenside surfaces are weaker then the surrounding clay where particles are randomly orientated. The friction angle on slickenside surfaces is called the residual friction angle. In highly plastic clays, this may be only 5 or 6 degrees, compared with peak friction angles of 20 or 30 degrees in the same clay. Slickensides develop most prominently in clay size dominated materials. Silt or sand size particles inhibit their formation. In some deposits, randomly orientated slickensides develop as a result of tectonic movements. These have less significance for slope stability as slickensides developed with an adverse orientation.

Decomposition of clayey rock fills

Clay shales and claystones excavated and used for fill usually compact to form a stable rock fill but through time, seepage of water into the rocks and numerous wet and dry cycles, the shales and claystones may slake and revert to chunks of disaggregated clay particles. These clays can then swell into voids in the fill losing strength and destabilising the fill.

Creep water under sustained loads

High plasticity clays tend to deform continuously under sustained loading and eventually fail under these sustained loads, even at shear stresses that are significantly less than the short term strength of the clays. Creep is exacerbated by cyclic freeze-thaw and wet-dry variations. When these cyclic variations are at their extreme, movement occurs in the downhill direction. These movements are permanent and not reversible under normal conditions. The long term results are ratcheting downslope movement that increases year after year. The end result is normally sliding on a continuous failure plane.

Leaching

Leaching involves a change in the chemical composition of pore water as it seeps through voids in any rock mass. A good example is the development of quick clays from marine clays as the salt is leached from the pore water of the clays. These quick clays have virtually no strength when disturbed.

Strain softening

Brittle soils are particularly subject to strain softening. This occurs after the peak stress has been reached in a stress-strain curve, the shearing resistances of brittles soils decrease with increasing strain.

Weathering

Weathering cause strength loss in rocks and indurated soils. The weathering process which involves physical, chemical and biological processes break the strong soils or rock into smaller pieces and the chemical and biological process change it into material with fundamentally different properties.

Cyclic Loading

Cyclic loading causes the bonds between soil particles to break and pore pressures to increase. Loose soils and soils with particles that are weakly bonded into loose structures are most vulnerable to loss of strength under cyclic loading. Loose sands may liquefy under cyclic loading where they virtually lose all strength and flow like a liquid (Duncan and Wright, 2005).

2.8.3 Factors controlling slope stability in southeast Viti Levu

The majority of landslides occur, or are initiated in the case of debris flows, by shear failure within the regolith. This takes place when the available shear strength of the slope material is exceeded by the in situ stresses. This may occur by a reduction in shear strength due an increase in pore water pressures, or by an increase in shear stress dues to static and dynamic loads. Static stresses may be increased by over-steepening of slopes due to removal of support by erosion and excavations, or by surcharge loads resulting from engineering structures or even landslide debris. Increased dynamic loads may be caused by earthquake shocks or by the movement of tall trees during high winds where stresses are transferred to the slope material by agitation of the root system. Once failure is initiated, the shape of the resulting shear surface is largely governed by the composition and thickness of the slope forming materials and the presence of pre-existing weaknesses or discontinuities in the slope.

Landsliding is closely related to high magnitude rainfall events is clearly indicated by the extensive slope failures which have occurred during major storms and cyclones. In the case of widespread debris flow development, exceedingly high rainfalls appear to be required before pore pressures are elevated sufficiently to induce failure an subsequent flow. During these high rainfalls, landslides occur on slopes of widely varying slope angle, but the location of these slides may in some cases be related to the nature of the surface regolith. Howorth et al (1980) noted that in the

Wainitubatolu catchment near Korovou village following cyclone Wally, five times as many slips occurred in highly weathered 'red' regolith as in less weathered 'brown' regolith, despite the latter having steeper mean slope angles (32 as compared to 27).

Earlier work on landslides in Viti Levu has revealed that the majority of failures have occurred or started on upper slopes at or near the slope crest. This tends to indicate that the sliding is caused by downward percolation of water rather than by a rising groundwater table, as the latter would have been likely to influence the lowest slope first (Vaughan (1985) in Greenbaum, 1995).

Residual red brown soil/regolith deposits (pedologically classified as latosols) are ubiquitous in the steep terrain most prone to landsliding, and are generally characterised by high void ratios and porosities, and low saturated unit weights. Based on soil and strength properties presented by Lawson (1993), limiting stability angles for 'wet' and 'dry' residual soils on slopes range from 12º-16º and 30º-35º respectively. Comparing to natural slope angles of 30º-38º in the southeast Viti Levu area, pore pressures approaching zero (equivalent to unsaturated conditions) must apparently be operating for stability to be maintained. High in situ shear strengths of the residual soils can be attributed to factors related to soil structure such as high porosities, permeabilities, infiltration capacities associated with the open textured bonded structure and the effect of soil suction. The combination of high in situ strengths and relatively high permeabilities in residual soils means that only prolonged high intensity rainfall can elevate pore water pressures to levels capable of inducing widespread landsliding; these are critical factors controlling landslide occurrence in southeast Viti Levu. Once sliding is initiated, a vast amount of soil water is available for the rapid translation of debris slides into debris flows.

Landslides in southeast Viti Levu can be triggered by sufficiently high rainfall on virtually all slopes, irrespective of slope angle and vegetation cover. Site specific occurrence of individual landslides appears to be controlled to a large extent by the variation in the material, geotechnical and hydrogeological properties of the regolith mantles (Greenbaum, 1995).

2.9 ENGINEERING PROPERTIES

2.9.1 Introduction

Soils can be visualised as a skeleton of solid particles enclosing continuous voids containing water and/or air. The range of stresses encountered in practice by individual particles of soil and water can be considered incompressible but the volume of the soil skeleton as a whole can change with the rearrangement of the soil particles through rolling, sliding with corresponding force changes occurring between the particles. This compressibility depends on the structural arrangement of the soil particles. Saturated soils are considered incompressible with volume reduction only possible through loss of water through seepage out from the void spaces in the soil. In dry and unsaturated soils, compression is almost always possible due to compression of air in the voids.

Shear stress can be resisted by the skeleton of solid particles by means of forces developed at the interparticle contacts. Normal stress can also be resisted through an increase in the interparticle forces and normal stresses in fully saturated soils can be resisted through an increase in pressure of the water filled voids (Craig, 1992).

2.9.2 Atterburg Limits

It is the acceptable convenient standard to express the plastic properties of soil materials in terms of plastic limit (Pl), liquid limit (LL) and plasticity index (PI) as first proposed by Atterburg in 1911. Allen (1942) defines:

Liquid Limit is the moisture content expressed as a percentage by weight of the oven dried soil at which the soil will just begin to flow when jarred slightly.

Plastic Limit is the lowest moisture content expressed as a percentage by weight of the oven dried soil at which the soil can be rolled into threads 3 millimetres in diameter without breaking into pieces. Soils which cannot be rolled into threads at any moisture content are considered non-plastic.

Plasticity Index is the difference between the liquid limit and the plastic limit. It is the range of moisture content in which a soil is plastic. When the plastic limit is equal to or greater than the liquid limit, the plastic index is recorded as 0.

These limits are determined according to procedures stated in NZS 4402 which is the 'Methods of testing soils for Civil Engineering purposes'.

Plastic Limit

The plastic limit is a measure of the water content which the particle surfaces can adsorb just slightly in excess of that can be fixed in a highly rigid condition and which does not separate the particles too much to greatly reduce the attractive forces between them. In addition to the oriented water directly adsorbed in the surfaces, there would be some pore water in a liquid or semi liquid condition enclosed in pores in such manner that it had little lubricating effect. The thickness of the water layers is difficult to estimate as is the amount of pore water at the plastic limit because the amount of adsorbing surface cannot be determined (Grim, 1962).

Liquid Limit

The liquid limit is the measure of the water which can be held with any substantial rigidity and which does not separate the particles, so that there is substantially no bonding force between them, plus the water at least in part in the liquid state, enclosed within the pores. The relative abundance of fixed water decreases and the pore water increases at the liquid limit as compared to the plastic limit (Grim, 1962).

Plasticity Index

The plasticity index is a measure of the range of moisture that can be held in excess of that with a high degree of rigidity and without the separation of the particles beyond that essential for a definite attractive force between them. It indicates something of the range of the moisture content with a relatively low degree of rigidity and which yields with slight applied force.

Weathering tends to increase the plasticity index value of clays. A change in the character of the adsorbed cations or even the development of different clay mineral composition such as motmorillonite form illite may be the explanation for this increase. It may also be a purely physical effect, in that the continual alternate wetting and drying may so weaken the bonds between particles and aggregates that they are so more easily dispersed (Grim, 1962).

2.9.3 Shear Strength

The shear strength of a soil is the internal resistance per unit area that the soil mass can offer t resist failure and sliding along any plane inside it (Das, 2002). Shear strength is measured by the shearing stress maximum displacement before failure. It is usually determined under conditions of increasing load pressure (Grim, 1962). For projects involving slope stability, the computation of the lateral pressure against the bracing of open cuts, bearing capacity of embankments, footings, or rafts in clay soils, the shearing resistance of the clays must be determined. If the water content of the clay will not change significantly during the period when the slopes are unsupported during the lifetime of the temporary bracing of the open cuts, or if the factor of safety of footings is minimum before the water content can decrease on account of the loading, the undrained shearing strength governs the behaviour of the clay (Terzaghi et al, 1996). The shear strength of a clay is made up to two parts, cohesion C_r, and the coefficient of internal friction tan ω_r according to the expression

$$T_f = c_r + \sigma_n' \, tan \, \omega_r$$

Where σ_n is the effective pressure normal to the shear plane. The cohesion is independent of the loading pressure and caused by a cementing or gluing force between particles. It is the shearing resistance after the removal of any loading pressure. In silts and sands with substantially no clay mineral content cohesion is zero. The factor of internal friction is due to the movement of one particle over another in general increases with increasing loading pressure (Grim, 1962).

2.9.4 Permeability

Soils are said to be permeable due to the existence of interconnected voids through which water can flow from points of high energy to points of low energy (Das, 2002). The abilities of soils, rocks and sediments to transmit and hold water constitute their most significant hydrologic properties.

Henry Darcy, a French engineer in the mid-nineteenth century designed an experiment to investigate the movement of water through sand. He established that the rate of water flow through a bed of a 'given nature' is proportional to the difference in the height of the water between the two ends of the filter beds and inversely proportional to the length of the flow path. He also determined that the quantity of the flow is proportional to a coefficient, K, which is dependent upon the nature of the porous medium. The resulting expression is called Darcys Law where discharge, Q, can be calculated as:

$$Q = -KA \, (h_A - h_B/L)$$

where

K = proportionality constant

A = cross sectional area of the pipe, A

h_A-h_B = is the difference in the head and

L = length is the flow length (Fetter, 1994).

While unconsolidated coarse grained sediments represent some of the most prolific producers of groundwater, clays on the other hand are used for engineering purposes such as lining solid waste disposal sites because of their low intrinsic permeabilities. The intrinsic permeability is therefore a function of the size of the pore opening. The smaller the size of the sediment grains, the larger the surface area the water contacts. This increases the frictional resistance to flow, which reduces the intrinsic permeability. The intrinsic permeability of well sorted sediments is proportional to the grain size of the sediment (Norris & Fidler in Fetter, 1994).

The intrinsic permeability of rocks is due to primary openings formed with the rock and secondary openings created after the rock was formed. The size of openings, the degree of interconnection, and the amount of open space are all significant.

Clastic sedimentary rocks have primary permeability characteristics similar to those of unconsolidated sediments. However, digenesis can reduce the size of the throats that connect adjacent pores through cementation and compaction. This could reduce permeability substantially without a large impact on primary porosity. Primary permeability may also be due to sedimentary structures such as bedding planes.

Weathering is another process that can result in an increase in permeability. As the rock is decomposed or disintegrated, the number and size of pore spaces, cracks, and joints can increase (Fetter, 1994).

2.9.5 Plasticity

Plasticity is an important characteristic in the case of fine grained soils, describing the ability of a soil to undergo unrecoverable deformation at constant volume without cracking or crumbling. Plasticity is due to the significant content of clay minerals or organic material. Dependent on its water content, a soil may exist in one of the liquid, plastic, semi-solid and solid states. Most fine grained soils exist naturally in the plastic state. The water contents at which the transitions between states occur, vary between soils dependent on the interaction between the clay minerals. Any decrease in water content results in a decrease in cation layer thickness and in increase in the net attractive forces between particles. For a soil to exist in the plastic state the magnitudes of the net interparticle forces must be such that the particles are free to slide relative to each other with cohesion between them being maintained. A decrease in water content also results in a reduction in the volume of a soil in a liquid, plastic or semi-solid state.

The upper and lower limits of the range of water content over which the soil exhibits plastic behaviour are defined as the liquid limit (LL) and the plastic limit (PL) respectively. The water content range itself is defined as the plasticity index (PI) derived by the formula: (Craig, 1997)

$$PI = LL - PL$$

2.10 LANDSLIDE MITIGATION OPTIONS

This section intends to discuss mitigative options that could be considered to stabilise the current slope instabilities at Wainigasau and Namuka-i-lau. The options elaborated on are based on the effectiveness of the remedial option in stabilising the area while at the same time keeping the cost of remediation to a minimal amount which is available for the construction of such measures.

2.10.1 Drainage

Drainage is the most frequently used means of stabilising slopes. Slope failures are quite often precipitated by a rise in the groundwater level and increased pore pressures so lowering groundwater levels and reducing pore pressures is the logical means of improving stability. Drainage is also relatively inexpensive in comparison with most other methods of stabilisation and large areas can be stabilised at low cost using a combination of drainage and other methods.

The first most important aspect of mitigating slope failure is understanding the nature of the slope failure. The following 5 factors need to be considered in choosing the methods that are technically feasible for the stabilisation of any slope.

Drainage improves slope stability by firstly reducing pore pressures within the soils and secondly by reducing the driving forces of water pressures in cracks thereby reducing the shear stress required for equilibrium. Functionality of any drainage system depends largely on its maintenance especially against erosion and siltation which disrupts surface drains and ditches and clogs underground drains. Siltation may be minimised by ensuring that drains are constructed by materials that satisfy filter criteria. Bacterial growth which also causes clogging of underground drains may be removed by flushing with chemical agents.

Surface Drains

Surface drains are used primarily to prevent water from ponding on the ground surface and to direct surface flow away from the slide area. This helps to reduce groundwater levels and pore pressures within the slide mass. Measures to improve surface drainage include:

1. Establishing lined or paved ditches and swales to convey water away from the site.
2. Grading to eliminate low spots where water can pond

3. Minimizing infiltration in the short term by covering with plastics, in the long term through vegetation and paving.

Vegetation works by increasing resistance to erosion by surface runoff and stabilizes the top foot or two of soil at the surface of the slope. In the long term evapotranspiration helps to lower the groundwater level.

Paving surfaces of slopes promotes runoff and impedes infiltration, but at the same time impedes evaporation and can lead to collection of water beneath the paved surface. Paving of clay slopes reduces the frequency of slides by minimising the seasonal wetting and drying cycles that can lead to gradual degradation in the shear strength of clays (Duncan and Wright, 2005).

Horizontal Drains

Horizontal drains are basically perforated pipes inserted in drilled holes in a slope to provide underground drainage. Horizontal drains generally slope upward to permit groundwater to drain by gravity. Horizontal drains vary in length up to 100 metres but longer drains can be used if needed. The drain pipes are commonly perforated or slotted PVC pipes although steel pipes were used in early installations. The drains are installed by drilling into the slopes using a hollow stem augur, inserting the drain pipe and removing the augur leaving the drain pipe in place. The holes are allowed to collapse around the drain pipe and no filters are used between the pipe and the surrounding soils.

Horizontal drains are usually installed from points of easy access for drill rigs and fanning out from this point to achieve broad coverage of the area. In a typical installation of horizontal drains, some of the drains will be productive and others non-productive but it is almost impossible to predict in advance which drains will produce significant flows. Flows usually decline in flow after installation and fluctuate seasonally through wet and dry seasons. Horizontal drains have found to be most effective when they are placed low in the slope, provided that the slope does not contain distinct layers of high permeability above the drains (Duncan and Wright, 2005).

Drainage Wells and Stone Columns

Vertical drains are used where soil strata of varying permeabilities are oriented horizontally and horizontal drains are not effective in intercepting seepage. The vertical drains which cross the layers are much more effective in intercepting seepages in these situations.

Drainage wells generally consist of vertical drains which are effectively large diameter holes (24 inch diameters are widely used) that are drilled and then filled with drain rock that satisfy filter criteria for the intercepted soils. These vertical drains are then tapped or intercepted by drilling from the base of the slopes to provide drainage by gravity. Vertical wells can also be drained by using deep pumps but the requirement for continual power ad pump maintenance makes this a more costly and less desirable alternative.

Stone columns function similarly to drainage wells provided that they have a low level outlet for the water that they collect. Once the drill holes are completed, the stone materials are compacted as they are placed in the drill holes and this has the further beneficial effect of increasing the strength of the surrounding soils by densification and increases in lateral stress.

Trench Drains

Trench drains are excavated trenches filled with drain rock that satisfies filter criteria for the surrounding soil. They are generally sloped to drain by gravity and may contain additional piping to increase flow capacity. Maintenance and inspection of the pipes are usually done through manholes installed for this purpose. The depths of the trench drains are usually governed by the requirement that the sides of the trench must remain stable without support until they are backfilled wit drain rock.

Finger or Counterfort Drains

Trench drains excavated perpendicular to a slope are called finger drains. Excavating trenches perpendicular to a slope does not affect the stability of the slope as much as would excavation of a trench drain parallel to the slope. The trench drain that connects the finger drains can be excavated and backfilled in short sections to avoid the destabilisation of the slope.

2.10.2 Excavations and Buttress Fills

Slope stabilisation can be increased by excavation through reducing the height of the slope or making the slope less steep. Flattening a slope or reducing its steepness reduces the shear stresses along potential sliding surfaces and increases the factor of safety. Excavation of the slope to improve stability more then always results in a loss of area at the crest of the slope. The three main ways are by excavating the top of the slope, excavating a bench or by flattening the slope.

There are two main types of buttress fills. A buttress of high strength well compacted material which provides strength and weight improving stability of the slope and the second type is a berm or gravity berm of uncompacted material at the bottom of the slope which provides weight and reduces the shear stresses in the slope, even though it consists of weak and compressible material. The effectiveness of berms is improved if it is placed on a layer of free draining material that allows drainage of water from the soil beneath.

2.10.3 Injection Methods

Injective methods are attractive because they can be implemented at relatively low cost. Their drawback is that it is difficult to quantify the beneficial effects. In addition when fluids are injected, the short term effect may be to make the slope less stable. The beneficial effects may be achieved only later when the injected material has hardened or has reacted with the soil to alter its properties.

Lime Piles and Lime Slurry Piles

Lime piles are drilled holes filled with lime. Lime slurry piles are drilled holes filled with slurry of lime and water. This method works most effectively for the stabilisation of clay and silty clay slopes. The interaction of the lime with the silts clays to strengthen the clays effectively improves the stability of the slope.

Cement Grout

Stabilising landslides by injecting cement grout has been used extensively and typical practice involves driving grout points about 2 metres apart in rows parallel to the track, the rows being 5 metres apart. The tips of the grout point are driven about 2 metres below the estimated depth of the rupture surface and grout is injected at each point.

2.10.4 Vegetation

Vegetation on slopes provides protection against erosion and shallow sliding. Root systems reinforce and bind soils to provide cohesion which improves stability against shallow sliding. Plant roots also reduce pore pressures within the slope by intercepting rainfall (reducing infiltration) and by evapotranspiration which effectively intercepts water flowing within the slope and diverting it to the surface., reducing pore pressures in the process (Duncan and Wright, 2005).

2.11 SYNTHESIS

- The most widely used and accepted version for the classification of landslides is that presented by Varnes (1978). The classification emphasizes the type of movement and the type of material involved. In the classification, the three material types are rock, debris and earth. There are five main types of movement namely falls, topples, slides, spreads and flows. Other landslide terms defined are the type of activity which include active, inactive, suspended, reactivated, dormant, relict and abandoned. A landslide can be advancing or retrogressing if the scarp is retreating further from the toe of the landslide. Rates of movements range from the very slow at a rate of 12 mm/yr to the very rapid at a rate of 5 m/sec. Water contents range from dry meaning no moisture content to very wet where enough moisture to flow like a liquid under low gradients.

- Landslides are caused by 3 basic broad types of landslide processes; those that increase shear stress, those that contribute to low strength and reduce material strength.

  The most common methods for increasing shear stress are by the removal of support, imposition of surcharges, transitory stresses from earthquakes or explosions and by uplift or tilting. A decrease in shear strength is achieved through increased pore pressure by rising groundwater levels as is frequently associated with intense prolonged rainfalls, cracking in rocks, swelling in overconsolidated and high plasticity material, development of slickensides, weathering, and cyclic loading.

- Determination of the engineering properties of the cut batter material is essential in determining its contribution to slope failure and the properties determined in this research include the determination of the Atterburg limits, the shear strength of the material which also gives us the friction angle and cohesion and permeability.

- The most effective landslide mitigation options are drainage, excavations and buttress fills, injection methods and vegetation.

CHAPTER 3

FIELD AND SITE OBSERVATIONS

3.1 INTRODUCTION

The Queens Road was constructed in the early 1960's and beginning in 1979, slope and cut batter failures along this national highway have caused considerable damage to road and building infrastructure and disruption to commuters.

The two sites investigated during this research are cut batters at Namuka i lau and Wainigasau (see Figure 3.1).

Figure 3.1 Location of Namuka i lau and Wainigasau Cut Batters

Both of these sites have had previous failure occurring in their cut batters. The southern Namuka i lau and northern Wainigasau cut batters failed during an intense and prolonged rainfall event in 1996. The northern Wainigasau cut batter has since failed again in 2003, 2007, 2008 and currently in 2009, following reconstruction of the cut batters in May of 2008. The northern Namuka i lau cut batter has had two failure events in 2009.

3.2 GEOLOGY

The area is dominated to the north by volcanics of the Lower Miocene Savura Volcanic Group and the older Mt. Gordon subgroup of the Wainimala Group. Overlying these unconformably are the Namosi andesites of the Medrausucu andesitic group from which the overlying Veisari and Namosi sandstones are derived. The Wainimala and Savura Volcanic groups are intruded by gabbros and tonalities of the Tholo plutonics and other upper tertiary aged intrusives of andesitic, dioritic and rhyolitic compositions.

3.2.1 Veisari Sandstone

The Veisari Sandstone is made up of 2 members, a sandstone unit made up of fine sandstones, pebbly sandstones and a polymict conglomerate. The second member is a volcanic conglomerate member which is derived extensively from the andesitic Mau volcano.

The Namuka i lau and Wainigasau sites are both made up of the andesitic conglomerates of the Veisari Sandstone unit. The conglomerates belong to the first member which is derived from the Namosi andesites. The conglomerates are extremely weathered and iron oxidation has transformed its colour to a red – reddish brown colour. The conglomerates are matrix supported and largely polymict with majority of the clasts derived from the Namosi Andesites.

The conglomerates are extremely weathered and though clast and bedding forms may be seen in the excavated and exposed surfaces, upon sampling the conglomerate disintegrate into a fine to very fine clayey silt material.

Bedding forms in the units are sub-horizontal to horizontal and strike orientations are difficult to measure because of the extreme weathering. Regional orientations of the unit strike east north east and have southerly dips of 5 to 15 degrees. There are indications of localised faulting and jointing exposed in the excavated faces of the cut batters. The microfaults measured along logging road excavations behind the Namuka i lau site have orientations from 348 to 022 with almost sub-vertical dips of 75 – 90 degrees.

The Veisari sandstones are restricted to the coastal lowlands and form a 45 kilometre strip along the coast starting at Mau. The strip along the coast is about 4 kilometres wide extending up from the coast to the foot hills of the Rama ranges.

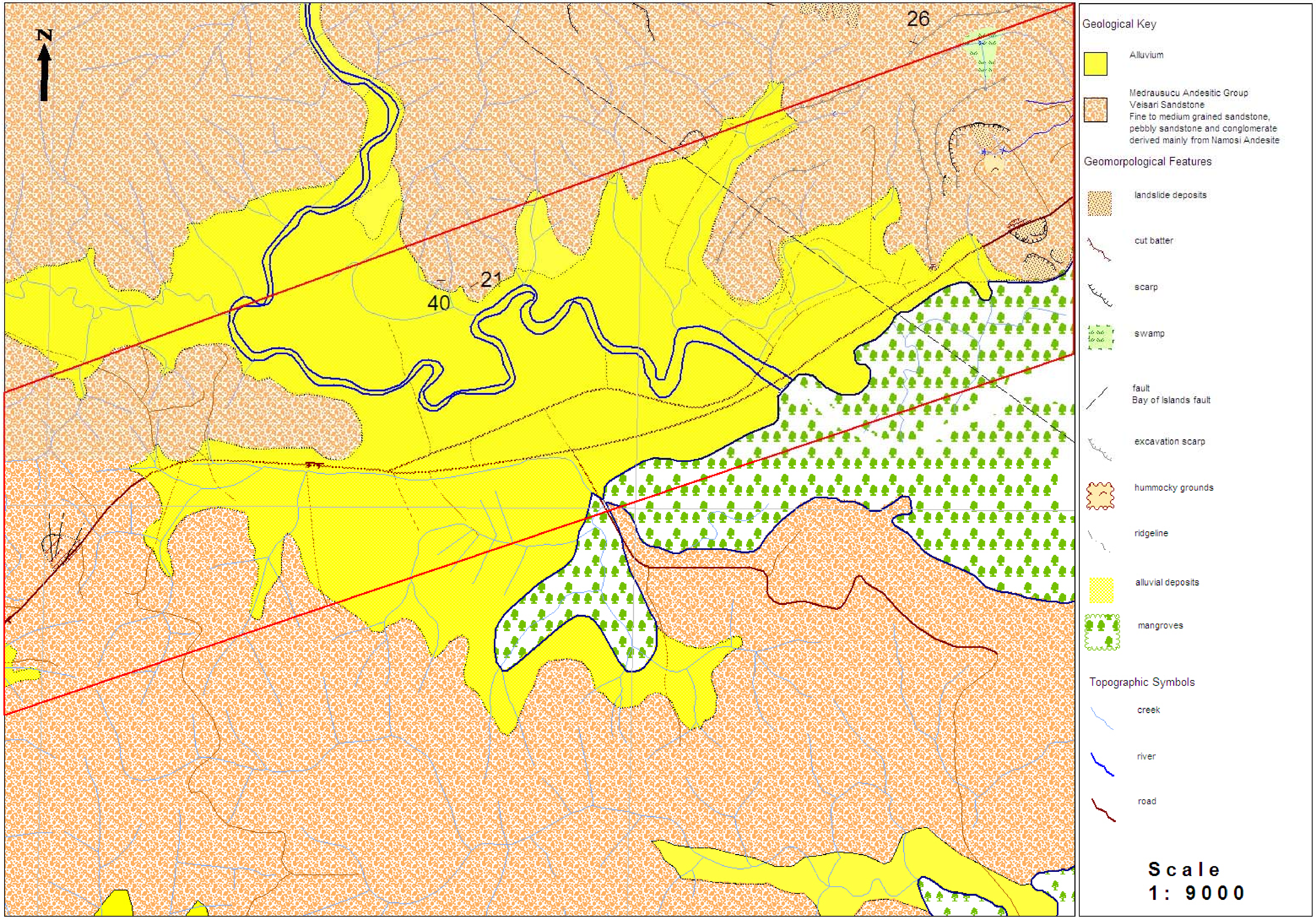


Figure 3.2 Geology of the Namuka i lau and Wainigasau corridor

3.3 GEOMORPHOLOGY

The general morphology of the Naumka i lau area consist of the Namuka harbour and estuaries that extend inland into alluvial plains of the Veisari and Wailekutu rivers. The Muaivuso peninsula and Suva peninsula are part of the coastal lowlands that extend to less than 100 metres in elevation. These coastal lowlands extend north into the higher elevations of the Rama and Medrausucu ranges and westward into the Navua plateau. The Rama ranges are rugged volcanic terrain underlain by gabbros of the Colo plutonic suite. The Navua plateau is a peneplain whose base level now stands 150 metres above sea level. Much of the plateau is underlain by tuffs and volcaniclastics of the Wainimala Group. Much of the upland relief of south east Viti Levu is carved from the Wainimala group volcaniclastic rocks in the south, the Navua basin sediments to the north and Suva marls in the east.

The general landscape shows a high density of deeply incised valleys by small creeks and streams and many grey cliffs, waterfalls and occasional caves. In the southeast, closer to the Suva peninsula, steep mudstone and sandstone landscapes gradually give way to hilly and rolling land in Suva Marl near the coast (Greenbaum et al, 1995).

The Rama ranges which comprise the Medrausucu group of volcanic rocks are largely dissected by creeks flowing in deeply incised valleys. These deeply incised valleys contribute significantly to the numerous landslides and slope failures that are seen in the Namuka i lau area. Aerial photo interpretations revealed large scale scarp features extending from the Rama ranges and extending down into the coastal lowlands, the largest scarp feature almost 6 kilometres long (see Figure 3.3).

Landslides in the higher ranges are rock slides and debris slides down the sides of the steep incised valleys. Due to the high regrowth rate of the subtropical vegetation, that covers the Rama ranges and coastal lowlands, many of the older landslides are unidentifiable using recent aerial photography. The recent landslides with fresh unvegetated scarps and deposits are easily identifiable and are included in the geomorphological map.

Deeply incised creeks and streams draining out from the Rama ranges feed into the Veisari and Wailekutu rivers. The creeks drain the coastal catchment which is situated between two major catchments, Navua catchment to the east and the Rewa catchment to the west. The creeks are densely distributed in the coastal catchment area and their dendritic forms become denser the further you move from the coastal and alluvial plains to the coastal lowlands and higher into the higher Rama ranges.

3.3.1 Namuka i lau

The coastal hills form a northern and western border to the Namuka i lau site (Figure 3.6). An older landslide scarp could have been reactivated during the construction of the cut batters at Namuka i lau and subsequent failure scarps and recent landslide scarps and deposits can be seen 300 metres north of the cut batter slopes at Namuka i lau. A small area of undeveloped hummocky ground are also visible signs left of the past events as much of the past slide deposits have been developed over as a residential subdivision. Smaller debris slides are also visible north of the Namuka i lau area occurring in the confines of steep creek incised valleys. The cut batter on the southern side of the main highway (Queens Road) has failed during a prolonged intense rainfall period in 2002. The cut batter was constructed on the northern face of a topographic knob with an elevation of 45 metres. The southern side of the knob also has two debris slide events which are occurred between 1986 and 2002 (post aerial photography of 1986 and older then event of 2002).

The Namuka i lau site is drained northwards and southwards by creeks which join towards the east of the site forming a creek which drains out to sea. Though no springs were mapped in the area, the Namuka i lau site has a swampy area at the confluence of the northward draining creek with the major creek. Much of the drainage at the site is controlled by cement v-drains on the side of the constructed roads.

The Namuka i lau site is bounded to the north, west and east by regional fault structures. The Namuka i lau site is bounded to the west by the Baby of Islands Fault 1 and to the east by the Bay of Islands Fault 2 and to the north by the Wailekutu fault. Mapping of the face of the cut batters revealed an absence of structural features in particular faults and joints.

3.3.2 Wainigasau

The Wainigasau site is located at the base of coastal hills that extend southwards towards the coast from the higher Rama ranges. The cut batters currently being investigated at Wainigasau are cut at the base of these southerly dipping slopes. The cut batter that is being investigated as part of this research has had multiple failures. An adjacent cut batter similar in size has had a single occurrence of failure at the base of the cut batter. The failure is a small sized failure less than 2 metres in length.

In contrast to the Namuka i lau site, the Wainigasau site has no evidence of failures of significant size in the vicinity of the cut batters both northwards (upslope) to the Rama ranges or laterally along the length of the but batters and adjacent coastal hills (see Figure 3.3).

The surrounding coastal hills are drained through a network of creeks that drain northwards to the Veisari River and southwards to the coast with no springs or occurrences of groundwater mapped in the area.

The Wainigasau site is bounded to the north by the Wailekutu fault trending approximately east/west, and to the east by the Bay of Islands fault 1 and to the west by the Muaivuso fault. The faults bound the site approximately 1.5 kilometres to the north, west and east. Three faults were mapped on the face of the newly excavated cut batters and have orientations trending 356, 006 and

030, and having sub-vertical dips. Indications of normal movement on the fault are minor (< 5 centimetres).

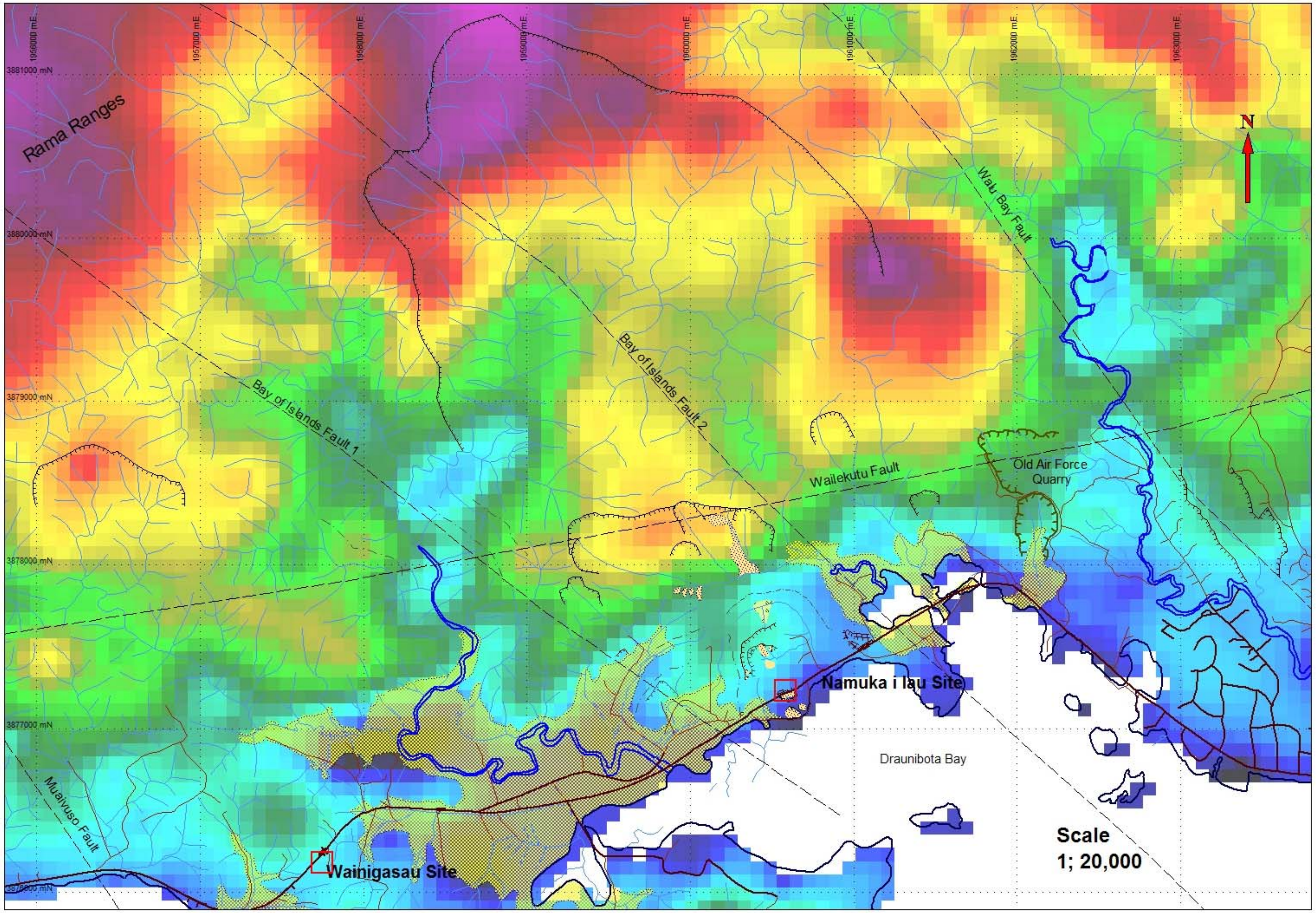


Figure 3.3 Fault structures overlaid over digital terrain data for the study area.

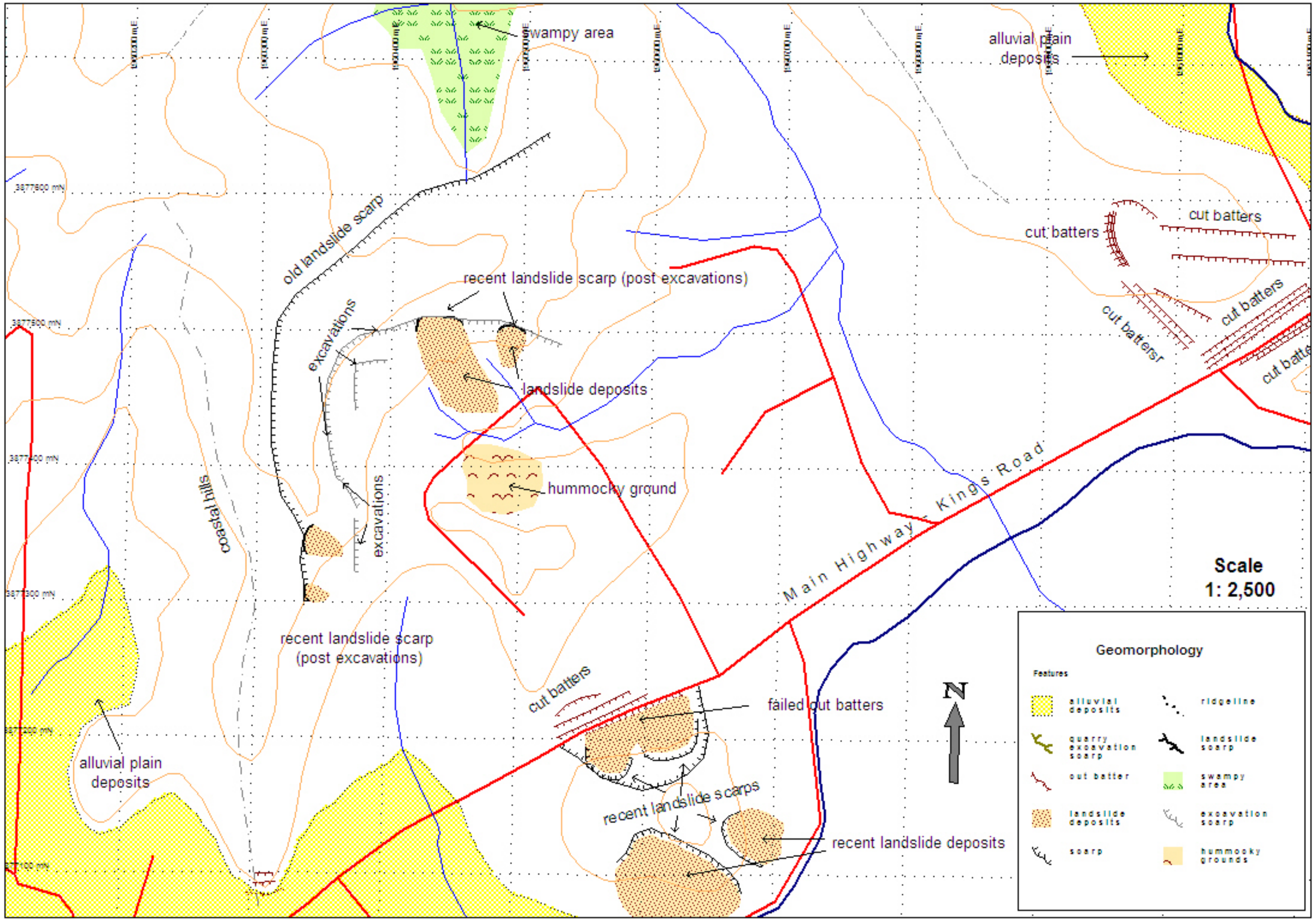


Figure 3.4 Geomorphological map of the Namuka i lau area

3.4 NAMUKA I LAU BATTER FAILURE

3.4.1 Site Description

The Namuka i lau site is made up of cut batters on either side of the Queens Road. The cut batters were formed through a hill (~42 metres height at summit) during construction of the current Queens Road. As seen in Figure 3.5 the previous Queens Road meandered around the small hill which was largely removed during the construction of the current Queens Road resulting in cut batter construction on either side of the current Queens Road. At initial construction cut batters on both the northern and southern sides of the road had 3 benches originally cut at around 50-60 degrees. The dimensions of the benches at Namuka i lau are summarised in the Table 3.1.

The cut batters are aligned northeast/southwest, with the northern cut batters sloping southeast and the southern cut batters sloping northwest (see Figure 3.6).

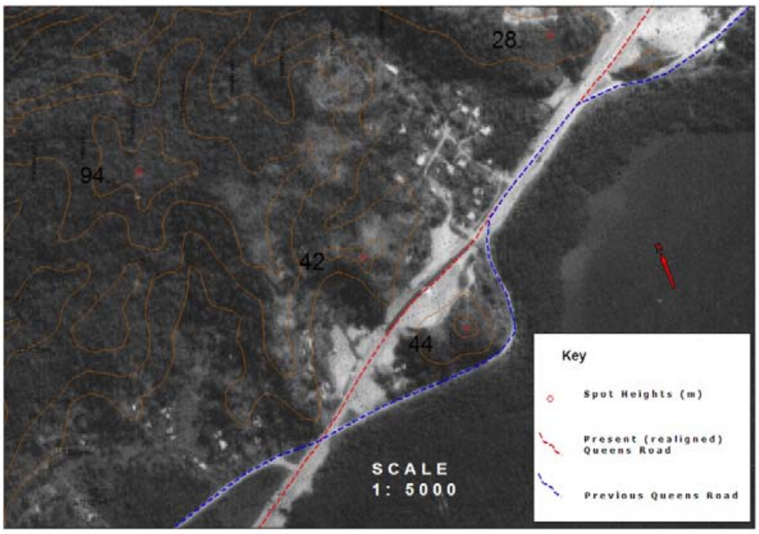


Figure 3.5 Realignment of the Queens Road

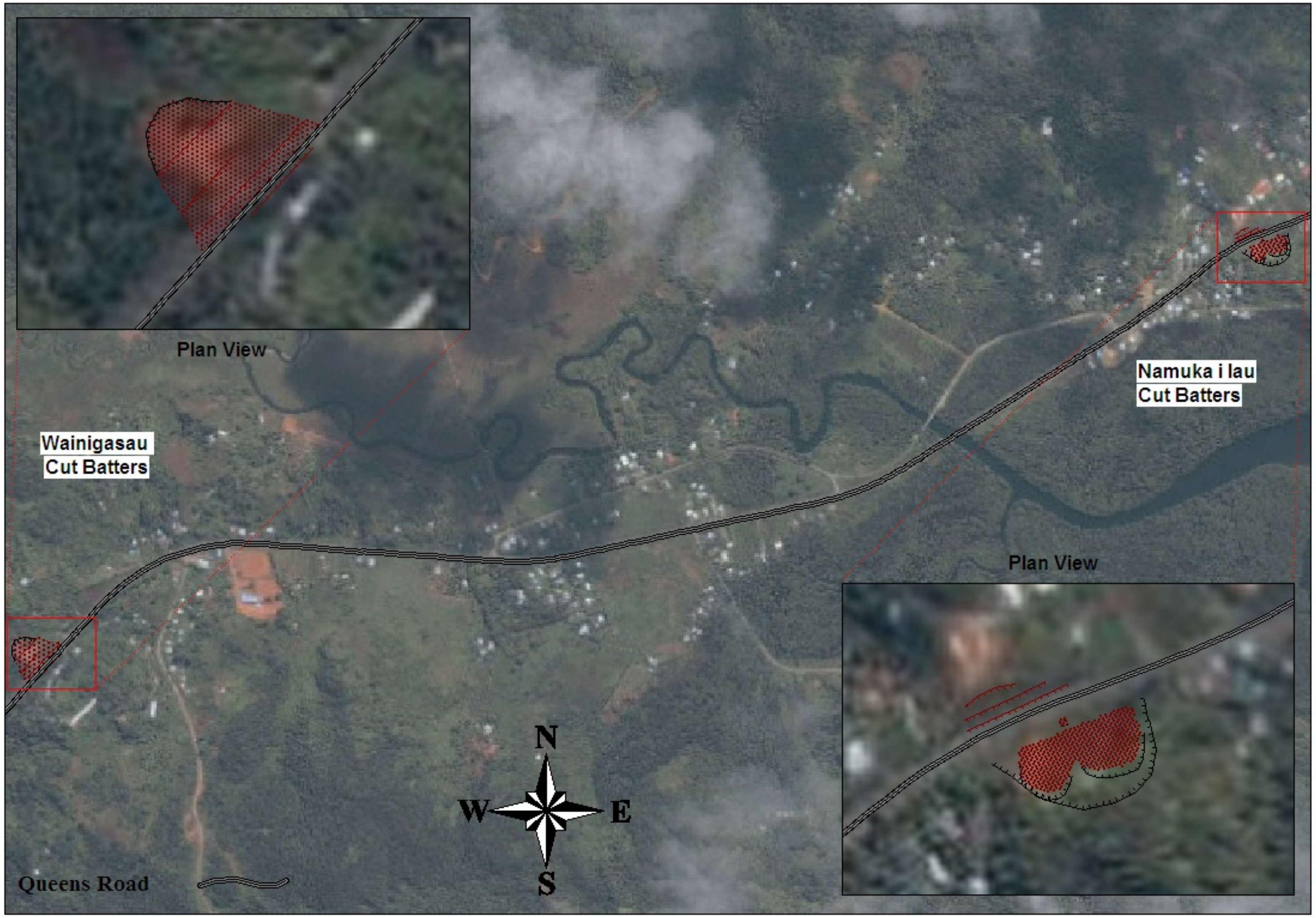
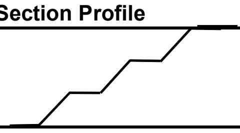


Figure 3.6 Plan view of cut batters at Wainigasau and Namuka i lau.

Namuka I lau Cut Batter Dimensions

Northern (inland) Cut Batter

	L (m)	H (m)		W (m)	Slope Angle			Average Slope	Section Profile
Bench 3	44.6	6.2	5.6		50	50	42	47.33	
Bench 2	70.5	8.6	9	1	48	52	55	51.67	
Bench 1	86.6	9.9	13	1	50	54	45	49.67	

Southern (coastal) Cut Batter

	L (m)	H (m)		W (m)	Slope Angle			Average Slope	Section Profile
Bench 3	failed	failed		failed	failed	failed	failed	failed	
Bench 2	failed	failed		failed	failed	failed	failed	failed	
Bench 1	120	failed		failed	failed	failed	failed	failed	

Table 3.1 Summary of dimensions for Namuka i lau cut batters.

3.4.2 Batter Configuration and History

The cut batters cut on the Northern Side of the road have 3 benches. The angles as summarised in the table above were originally cut at an angle of approximately 50 º but have since started to degrade since construction.

The cut batters are largely intact and undisturbed but small (< 2 metres depth) failures have begun to occur in June of 2009. The failure of the cut batter has shown retrogressive movement since the initial failure in June and subsequent failures have occurred in August and September of 2009. The failures are restricted to the lowest bench adjacent to the Queens Road but signs of retrogressive movement are currently being seen.

Subdivision development has occurred above the cut batters post construction with housing construction being carried out post 1998 (last aerial photography). This has provided additional loading to the cut batters (see Figure 3.6).

Since construction the cut batters in 1962, the cut batters on the southern side of the road have since failed and destroyed the benching structures except for approximately 25 metres of bench on the western end of the cut batters (see Figure 3.7). There have been numerous past failures on the cut batter resulting in a large scarp feature that has joined the head scarps of the past failures (see Figure 3.7).

Figure 3.7 Namuka i lau southern bench structures destroyed except for 25 metres on western end of cut batter structures

3.4.3 Geology and Profiles

The cut batters are cut in highly weathered polymict conglomerates. The conglomerates are interbedded with mudstone and sandstone layers. The polymict conglomerate clasts are volcanic in origin and original rock types are difficult to determine due to the high degree of weathering. The mudstone and sandstone beds vary in thickness from thinly laminated to medium thickness (< 10 centimetres). The conglomerate is matrix supported with matrix material consisting of medium to fine grained lithic material.

The cut batter surfaces were void of structures such as faulting and jointing but extensive rilling structures were seen in the failed southern cut batters. These rilling structures were eroded by excessive surface runoff.

The slopes of the 3 benches for the northern cut batters are cut at approximately 50 degrees and slope angles are preserved laterally for the entire length of the benches.

The southern cut batter failed in 1996 but no rehabilitation has occurred on the site except for the placement of 2 rows of gabion baskets at the foot of the failed cut batters to act as a debris barrier in case of further failures.

Groundwater

No evidence of groundwater was mapped around the site of the northern cut batters, with the absence of any creeks or springs. Surface runoff created during periods of high precipitation flow freely over the surface of the cut batters and down the face of the benches.

A concrete v-drain lies at the foot of both the northern and southern cut batters adjacent to the Queens Road to capture surface runoff from the cut batter and the road and discharged into the highway stormwater drainage system.

There were no creeks or springs mapped on the southern cut batters, and drainage of the cut batter is restricted to a v-drain at the toe of the slope adjacent to the Queens Road. Surface runoff during periods of precipitation flow freely down the face of the slope causing surface rilling.

Vegetation

The northern cut batters are vegetated with regrowth consisting mainly of scrub, ferns, reeds and small trees. Vegetation covering the southern cut batters is sparse. The failed cut batters and the remaining benches being vegetated mainly with scrub, reeds and fern vegetation. Young tropical forest vegetation covers the top and surrounding slopes of the knob onto which the cut batters are constructed.

3.5 WAINIGASAU BATTER FAILURE

3.5.1 Site Description

The Wainigasau cut batter was constructed as part of the current Queens Road during the 1960's. The cut batter is cut between the coastal hill ranges of the southwest Viti Levu coastline and the Waiqanake Peninsula. The cut batter of concern is on the northern (inland) side of the Queens Road and is cut into the toe of a small hill with a spot height of 85 metres. The cut batter was originally formed as 3 benches (as determined from past aerial photos) but original slopes for the benches have been destroyed during failure of the cut batters beginning in 1996, and continuing in 2003, 2007 and 2008 and currently in 2009. In May of 2008, the cut batter after numerous failures was recut into 3 new benches at slopes of 60 degrees, and dimensions of which are summarised in Table 3.2 below. These benches have since failed beginning September 2008 and continuing through to July of 2009.

The cut batter is aligned northeast/southwest with the northern (inland) benches sloping southeast (see Figure 3.6).

Wainigasau Cut Batter Dimensions

Northern (inland) Cut Batter

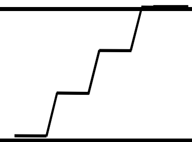
	L (m)	H (m)		W (m)	Slope Angle			Average Slo	Section Profile
Bench 3	62	9.2	9		60	60	60	60.00	
Bench 2	66.5	7.1	5	3	60	60	60	60.00	
Bench 1	74	4.8	9	3	60	70	60	63.33	

Table 3.2 Summary of dimensions for Wainigasau cut batters.

3.5.2 Batter Configuration and History

The northern cut batters were constructed as part of the Queens Road in the 1960's and were initially cut as 3 benches. The initial bench slopes are estimated at about 60 degrees but have since been reconstructed after multiple failures. The initial benches have had multiple failures beginning first as small (<2 metres) debris slides on the lowest bench adjacent to the Queens Road and later actively retrogressing into a larger debris slide extending up to the second bench. A list of major failure events at the northern Wainigasau bench are as follows:

- A major event occurred in the storm season of 2002 during the month of December, when the cut batter failed with the scarp of the slide being at the height of the second bench. The amount of debris which blocked the Queens Road for half a day was estimated at between 4000 – 6000 cubic metres.
- The failure spread laterally and retrogressively to the third bench through the years 2003 – 2008 when a second major failure occurring in January of 2008 caused the rest of the cut batter to fail displacing the second bench and third benches by up to 2 metres.
- Reconstruction of the entire cut batter began in May 2008, reconstructing the 3 benches at 60 degree slopes. Half of the second bench failed during construction and a single bench now extends down from the third bench to the first bench.
- The second bench terminates laterally in the middle of the cut batter.
- The first bench of the cut batter failed again in September of 2008 and again in July of 2009.

A photographic log of the above events is presented in Figure 3.8.

The cut batters on the southern side of the road are essentially the termination of the slope of the spur into which the cut batters on the northern side have been cut. No failures have occurred on this southern cut batter.

3.5.3 Geology and Profiles

The cut batters are cut in highly weathered conglomerates. The conglomerates are interbedded with mudstone and sandstone layers. The polymict conglomerate clasts are volcanic in origin and original rock types are difficult to determine due to the high degree of weathering. The mudstone and sandstone beds are thinly laminated with beds less than 20 centimetres thick. The conglomerate is matrix supported with matrix material consisting of medium to fine grained lithic material.

The northern cut batters are transacted by microfaults/fractures possibly related to the Muaivuso and Bay of Islands faults which lie to the east and west of the cut batters. Orientations of the microfaults/fractures are similar to the Bay of Islands and Muaivuso faults having strike directions between 348 and 022 with subvertical dips.

The faces of the benches show extensive rilling structures eroded by excessive surface runoff down the faces of the benches during periods of precipitation. Vertical excavation furrows down the faces of the benches have also provided pathways for surface runoff to flow and deepening of these furrows through erosion have created vertical fissure like structures weakening the structure of the bench faces.

A combination of the microfaults/fractures, rilling structures and vertical fissure like structures to create possible future failure structures.

The slopes of the 3 benches are cut at approximately 60 degrees. The second bench terminates at the centre of the cut batter as the other half of the bench has failed. For the eastern half of the cut batter, the third bench now extends down at an angle of 60 degrees to the first bench.

The southern cut batter is a single bench less than 2 metres in height.

Groundwater

There were no springs or creeks mapped at the area but surface runoff have contributed significantly to surficial features of the cut batters and hydrological conditions of the cut batters.

The surfaces of all 3 benches have significant evidences of surface rilling caused by excessive surface runoff. The surface runoff has eroded parallel rilling structures sub vertically down the face of the third and second benches.

Microfractures and minor jointing in the weathered rock mass have been enlarged and eroded further by surface runoff and have combined with the rilling structures to form cross cutting fissure like structures that seem to be the precursors to the formation of failure surfaces in the cut batters.

Excavated drains at the foot of the second and third benches are filled with finer silt and clay fractions but excavations of the drains are incomplete as no outlet for the drains were excavated so water collected in the drains have eroded channels at the edges of the cut batters. The excavated drains at the top of the third bench are also filled with finer fractions of silt and clays and evidence of ponding is seen adjacent to the sediment clogged drains. Cracks can be seen at the edge of the third bench caused by erosion from excessive surface runoff and repeating wet and dry cycles.

Concrete drains on the eastern edge of the cut batters has been blocked and destroyed by previous failures and reconstruction of the cut batters. The concrete drain at the foot of the cut batter is filled and blocked by debris deposits of the last failure.

For the southern cut batters, there are no mapped springs or creeks and surface runoff is minimal due to the dense tropical shrub and bush that has vegetated the cut batter. A concrete v-drain at the front of the cut batter drains the runoff from the Queens road and discharges into the highway stormwater drainage system.

Vegetation

The southern cut batter is completely covered with young tropical vegetation consisting of young trees and thick undergrowth. The dense vegetation and abundance of root structures from the thick tropical vegetation has minimised the destabilising effects of surface runoff. The abundant root structures also increase stability by holding the soil particles together.

The northern cut batters were sparsely covered with scrub, fern vegetation and young pine trees before the failure of 2003 but since the reconstruction of the benches in 2008 the cut batters are completely unvegetated with only shrub and young trees atop of the third bench.

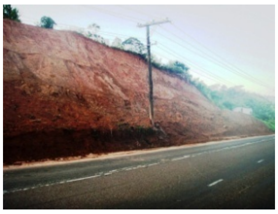
Date	**Photo Log**	**Description of events**
December 2002		Initial cut batter failure Head scarp at 2^{nd} bench
January 2008	  	Failure migrating laterally to eastern limit and retrogressing to 3^{rd} bench
May 2008	  	Subsequent failure leading to reconstruction of cut batters into 3 benches at 60 degree slopes
September 2008	  	Failure of reconstructed benches 1 and 2
August 2009		Failure of reconstructed bench 1

Figure 3.8 Photographic log showing the progression of failures at Wainigasau

3.6 SYNTHESIS

- The northern Namuka i lau cut batters consist of 3 benches. The benches are approximately 9 m high and 1 m wide. The batter faces have an average slope of 50º. No evidence of groundwater were mapped at the site but excess surface runoff flows are captured at the base of the batters and drained into the highway stormwater drainage system. Vegetation is sparse and small failures have occurred in 2009. The southern cut batters have identical conditions but have failed post construction except for 25 m remaining on the western end of the batters.
- The northern Wainigasau batters have failed repeatedly since 1996. They were reconstructed in 2008 into 3, 10 m high benches sloping at 60 degrees. The benches are 3 m wide and unvegetated. The batters have microfaults and jointing structures exposed in the excavated faces and groundwater has eroded extensive riling structures in the second and third benches. Excavated drainage is blocked with fine silt material and installed concrete drains were destroyed during reconstruction of the batters.
- The geology of the two sites is composed of a highly weathered andesitic polymict conglomerate that belongs to the Veisari Sandstone unit. Iron oxidation is high as seen by its reddish appearance and clast and matrix material have been highly weathered to silt and clay material that disintegrate easily.
- Geomorphological conditions for the two sites are markedly different. The Namuka i lau site has evidence of previous landslide activity including scarps and hummocky ground. To the north in the deeply incised valleys, we see evidence of debris slides. The Wainigasau site did not show much evidence of previous landslide activity except for the previous cut batter failures.

CHAPTER 4

LABORATORY STUDIES

4.1 INTRODUCTION

During the course of field investigations, samples of the rock material from the Namuka i lau and Wainigasau site were collected for laboratory studies. The results were used in conjunction with our observations from field investigations to develop a geotechnical model and analyse the stability of the cut batters. Results of the laboratory studies are discussed in the following text while laboratory techniques and test data are found in Appendix A.1 – A.7. A discussion of the results is presented at the end of each of the analysis conducted.

4.2 X-RAY DIFFRACTION ANALYSIS

The material at the Wainigasau cut batter was found to be made up of a highly weathered polymict conglomerate. The highly weathered polymict conglomerate had weathered completely to silt and clay fractions. The purpose of the x-ray diffraction (XRD) analysis was to determine the types of clays that were contained in the cut batter material, and also to estimate the amounts of the different clays and other mineral compositions found in the cut batter material.

Ten representative samples of the differing clasts of the polymict conglomerate and one sample of the matrix from the Wainigasau site were sampled for XRD analysis. The Wainigasau site was sampled as the recent excavations had exposed fresh rock surfaces that had not been exposed to current climatic conditions, post construction of the cut batters. The eleven samples were instantly placed in plastic bags upon sampling, and heat sealed on return to the office to minimise drying out of the samples. It was noted that some clay compositions alter their mineralogies upon drying, and care was taken to minimise the chances of this occurring prior to conducting XRD analysis of the samples.

The samples were sent by airfreight to the University of Canterbury where they were submitted upon arrival for XRD testing. Small amounts of the clasts and matrix were taken and initially tested. Satisfactory results in terms of mineralogy were obtained from this method so separation through 9Ø suspension to seperate clay fractions for further testing was not necessary.

4.2.1 Methodology

A very small piece (5 grams) was placed in a mortar and immersed in ethanol and then ground using the pestle to a fine powder. This was then applied to a small slide, using a disposable pipette, with care being taken to concentrate on 2/3 of the slide, leaving 1/3 free for identification marking and placement in the Diffractometer. The sample was then left to dry and then placed in the Diffractometer for analysis. Mortar and Pestle were washed and dried, and the disposable pipette discarded. The Diffractometer used for this exercise was the Philips PW1820/1710 X-ray Diffractometer system.

4.2.2 Results

A summary of results is summarised in Table 4.1 below and the full result printouts are attached in Appendix A.1.

Sample	Component	Percentage Estimate	Scan line colour
Clasts 1	Quartz Kaolinite Hematite	95 5 <1	Blue Yellow Red
2	Quartz Kaolinite Hematite	80 15 5	Blue Yellow Red
3	Quartz Kaolinite Hematite	65 20 15	Blue Yellow Red

4	Quartz	85	Blue
	Kaolinite	15	Yellow
	Hematite	<1	Red
5	Quartz	100	Blue
	Kaolinite	<1	Yellow
	Hematite		Red
6	Quartz	90	Blue
	Kaolinite	10	Yellow
	Hematite	<1	Red
7	Quartz	90	Blue
	Kaolinite	<1	Yellow
	Hematite	10	Red
8	Quartz	65	Blue
	Kaolinite	25	Yellow
	Hematite	10	Red
9	Quartz	100	Blue
	Kaolinite	<1	Yellow
	Hematite	<1	Red
10	Quartz	95	Blue
	Kaolinite	5	Yellow
	Hematite	<1	Red
Matrix 11	Quartz	95	Blue
	Kaolinite	5	Yellow
	Hematite	<1	Red

Table 4.1 Summary of results for XRD analysis for clasts and matrix samples from Wainigasau

4.2.3 Discussion

The results of the XRD analysis clearly indicate that the only clay mineralogy found in the clasts and matrix tested is Kaolinite. The quartz signature makes it difficult to accurately determine the exact percentages of the mineralogies found in the clasts and matrix as the quartz signature tends to overprint those of the other mineralogies present. The presence of significant amounts of quartz though is expected for the high level of weathering seen in the material being tested. In highly weathered rocks quartz tends to be preserved, since it is relatively resistant to weathering processes.

In comparison, feldspars in rocks tend to alter quite easily to clay minerals during weathering processes. Kaolinite is the weathering product of feldspars in the original rock and presence of kaolinite also alludes to the weathering intensity. Potter, Maynard and Depetris (2005) determined that kaolinite type clays predominate between the tropical coasts of Africa and Brazil because of the weathering intensity and maximal leaching present in the humid tropics of these regions.

The presence and percentages of hematite determined by the XRD analysis is consistent with the reddish colour of the material being tested. The hematite is most probably the alteration products of ferromagnesian minerals and iron sulphides present in the original rock.

4.3 DETERMINATION OF WATER CONTENT AND BULK UNIT WEIGHT (WEIGHT DENSITY)

In understanding slope failures, basic properties to consider in determining material strength properties are the natural water content, density of the material, and the unit weight. The combined results of the determination of the water content and bulk unit weight are presented together following the methodologies. The full calculations can be found in Appendix A.2.

4.3.1 Determination of Water Content Methodology

The water content was determined according to NZS4402 (1986) Test 2.1. The method covers the determination of the water content of the sample as a percentage of its dry mass. The initial steps involved the weighing the container before the addition of the sample to 0.001 grams. After addition of the sample to the container, the container and sample were then weighed again before drying of the sample was carried out. Drying of the sample was carried out in a drying oven heated to 105°celcius for a period of 24 hours. After the drying period, the container and dried sample were then weighed again, and the water content, w, calculated using the following formulae:

$$\boldsymbol{w = \frac{m_2 - m_3}{m_3 - m_1} \times 100}$$

where m_1 = weight of container (g)

m_2 = weight of container and wet sample (g)

m_3 = weight of container and dried sample (g)

4.3.2 Determination of Bulk Unit weight Methodology

In determining the bulk unit weight of the samples, before the samples were weighed and dried, they were cut into cubes of measurable sides so their volumes could be calculated and used for the calculation of the bulk unit weights of the samples.

The bulk unit weight (partially saturated), γ_b, of the samples were determined using the following formulae:

$$\gamma_b = \frac{\text{total weight}}{\text{total volume}} = \frac{m_t g}{v_t}$$

where m_t = total mass (g)
v_t = total volume (m^3)
$g = 9.81\ m/s^2$

4.3.3 Results

The results for determination of water content and bulk unit weight for the Namuka i lau and Wainigasau sites are summarised in Table 4.2 below:

Property	Namuka i lau	Wainigasau
Water Content, w, (%)	47.2	44.9
Bulk Unit Weight, γ_b, (kN/m^3)	14.5	15.8

Table 4.2 Results of water content and bulk unit weight determination

4.3.4 Discussion

Typical densities and unit weights for different soil types are summarised in Table 4.3 including the results from Namuka i lau and Wainigasau.

Soil Type	Bulk density (Mg/m^3)	Unit Weight (kN/m^3)
Sand and gravel	1.6 – 2.2	16 – 22
Silt	1.6 – 2.0	16 – 20
Soft clay	1.7 – 2.0	17 – 20
Stiff clay	1.9 – 2.3	19 – 23
Peat	1.0 – 1.4	10 - 14
Weak intact rock (mudstone, shale)	2.0 – 2.3	20 - 23
Namuka i lau	**1.5**	**14.5**
Wainigasau	**1.6**	**15.8**

Table 4.3 Typical densities and unit weights (adapted from Barnes, 1995) and including values determined from Namuka i lau and Wainigasau sam

The unit weights calculated for the Namuka i lau and Wainigasau samples are lower than typical values calculated for weak intact rock but are comparable to silts and soft clays. This can be explained by the non-homogeneous nature of the material being tested. The material is a highly weathered polymict conglomerate, with most of the original rock material altered into kaolinite clays, hematite and fine silt and clay fractions. The material also consists of interbedded mudstone and sandstone layers. The variability in bulk density and unit weight is also dependent on the location of the sample site and the composition of the material being tested.

4.4 ATTERBURG LIMITS DETERMINATION

4.4.1 Methodology

A 4 kg sample of the weathered polymict breccia was submitted to the Geotechnical Laboratory of the Department of Roads in Fiji for sieve analysis, determination of plastic limit (PL), liquid

Limit (LL) and plasticity index (PI). The Geotechnical Laboratory was not able to carry out a complete grain size analysis. Grainsize separation of the coarse and medium fractions were carried out by sieve separation but the separation of the finer fraction, (less than 63 μm) usually carried out using pipette analysis was not so the results presented are those only for the coarse fraction of the samples submitted. The Geotechnical Laboratory uses British Standards and the full test results can be found in Appendix A.3.

4.4.2 Liquid Limit

The liquid limit was carried out according to British Standards (BS1377:1990) using the definitive cone penetrometer method. The test consists primarily of a 30ºcylindrical cone, 80 grams in weight with a smooth, polished surface allowed to drop freely into a cup of prepared sample close to its liquid limit.

The penetration of the cone and the moisture content of the soil are measured for subsamples and a graph of cone penetration versus moisture content is plotted for the samples tested. The liquid limit of the sample is taken as the moisture content at a penetration of 20 millimetres.

4.4.3 Plastic Limit

The Plastic Limit similarly was carried out using British Standards (BS1377:1990). The sample to be tested is prepared by getting it as close as possible to its plastic limit by moulding it in the palm of the hand of the operator until a smooth thread of about 6 millimetres in diameter and about 50 millimetres in length can be formed. The thread is placed on a glass plate and rolled beneath the fingers of one hand in a backward and forward motion applying just enough pressure to reduce the thread to a diameter of 3 millimetres. The test is complete when the thread shears both longitudinally and transversely. The thread is now completely transferred to a weighing bottle to determine its moisture content. Sub samples are identically tested and the average of the moisture content gives us the plastic limit of the sample.

4.4.4 Plasticity Index

The Plasticity Index, PI, is the difference between the liquid limit and the plastic limit determined by the formula: **PI = LL-PL**

4.4.5 Results

The results for liquid limit, plastic limit and plasticity index are summarised in Table 4.4 below:

	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
Namuka i lau	68	51	17
Wainigasau	76	61	15

Table 4.4 Atterburg Limit results for Namuka i lau and Wainigasau

4.4.6 Plasticity Chart

The Atterburg Limits are useful in identifying the type of clay mineral present by plotting the plasticity index and liquid limits on a Plasticity Chart (see Figure 4.1). Samples plotted above the A-line are predominantly clays (C) and those plotting below are predominantly silts (M). Organic soils also plot below the A-line and are given the symbol, O.

Varying degrees of plasticity are also distinguished according to their liquid limits (LL) as shown in Table 4.5 below:

Plasticity (Symbol)	Low (L)	Intermediate (I)	High (H)	Very High (V)	Extremely High (E)
Liquid Limits	< 35%	35 – 50%	51 – 70%	71 – 90%	>90%
Namuka i lau (MH)			68		
Wainigasau (MV)				76	

Table 4.5 Plasticity classifications for Namuka i lau and Wainigasau

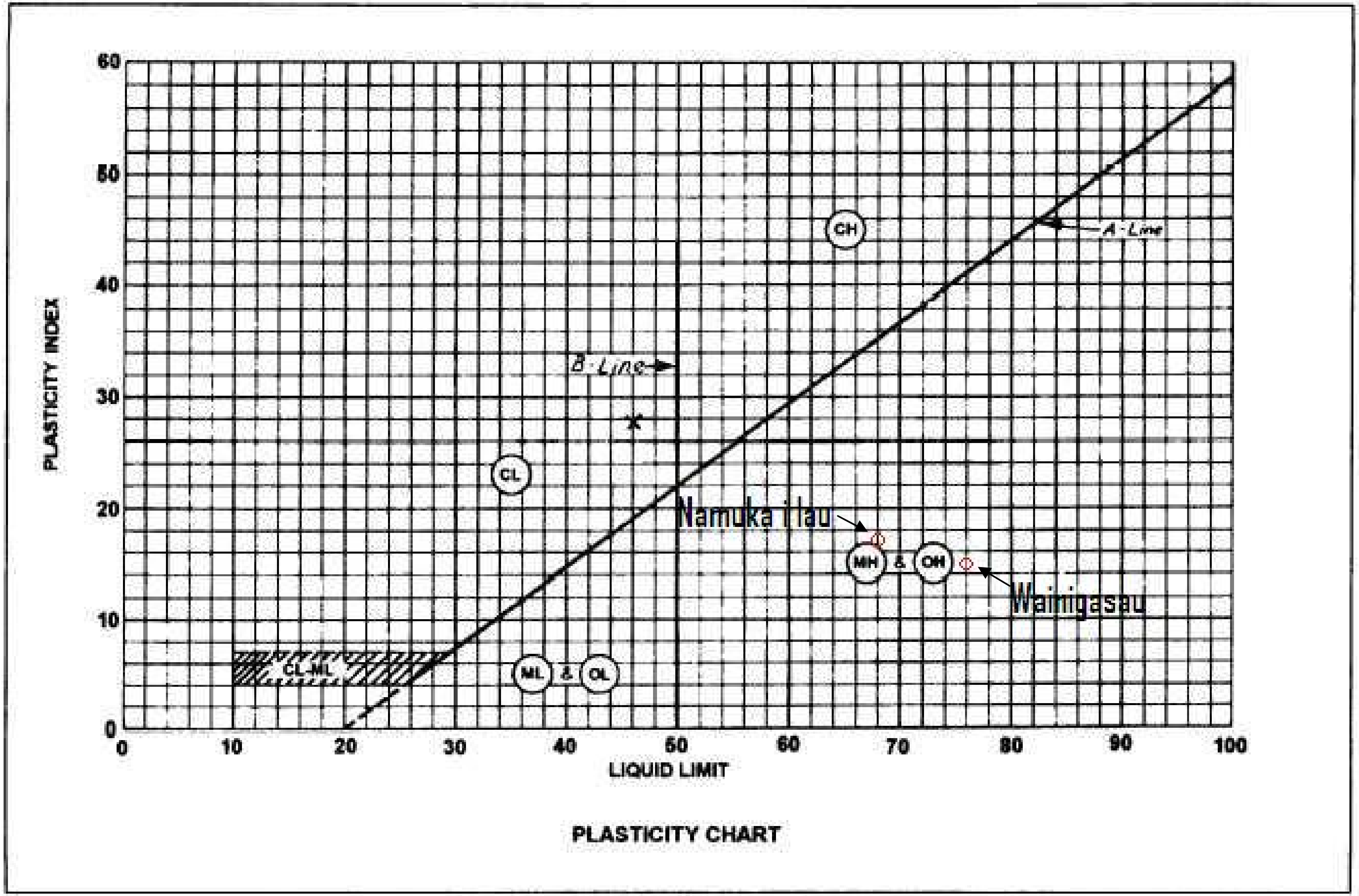


Figure 4.1. Plasticity Chart Plot for Namuka I lau and Wainigasau samples.

4.5 GRAINSIZE ANALYSIS

4.5.1 Methodology

The grainsize analysis was carried out by the Public Works Department Geotechnical Laboratory in Fiji. The results received from the Geotechnical Laboratory revealed only sieve analysis were carried out on the samples. The smallest sieve size was 63 micrometres so the silt and clay fractions were not able to be separated out and the results shown below show a combined percentage of silt and clay particle sizes. A pipette analysis of the fine silt and clay fractions would have completed the size analysis result which unfortunately was not able to be conducted due to an unavailability of appropriate equipment and expertise. Clast and matrix samples sent for XRD and shear testing were not of sufficient volumes or representative of the material being tested to allow for an accurate pipette analysis. The results presented below show a cumulative percentage composition for the samples tested.

4.5.2 Results

The full test data results for the sieve analysis can be found in Appendix A.4. The size fractions are summarised in Table 4.5 below and in Figure 4.2 and Figure 4.3:

Sieve Size	Namuka i lau (%)	Wainigasau (%)
Gravel (> 4750 µm)	0	9.15
Sand (75 µm - 2360µm)	56.32	51.55
Silt and Clay (< 63 µm)	43.68	39.30

Table 4.6 Summary of results for grainsize analysis of Namuka i lau and Wainigasau sample

The results as plotted below show a Silty Sand or Clayey Sand material tested at Namuka i lau dependent on the amounts of silt to clay fractions had suspension separation been carried out on the sample. An appropriate name for the material could be **Silty Clayey Sand**.

The results for Wainigasau plotted below show a small percentage of gravel (coarser then 4750 micrometres) sized material of 9.15 percent. Standard naming practice would name the material tested from Wainigasau as a **Silty or Clayey, gravelly Sand**, depending on the percentages of the silts and clays if suspension separation had been conducted on the sample.

4.5.3 Discussion

The size distribution plots show that the material is moderately well graded as seen by the curve distribution but the lack of fine fraction data limits any computation of grading percentage . Both the samples show at least 50 percent sand fraction and approximately 40 percent silt and clay size indicating that the distribution of the grainsizes is within the sand and silt fraction size. The 9 percent gravel fraction seen in the Wainigasau sample is expected because of the clasts of the polymict conglomerate, which have not have been as highly weathered as the rest of the material. This is seen in the XRD results also, a major component of these gravel fraction being made up of the resistant quartz material.

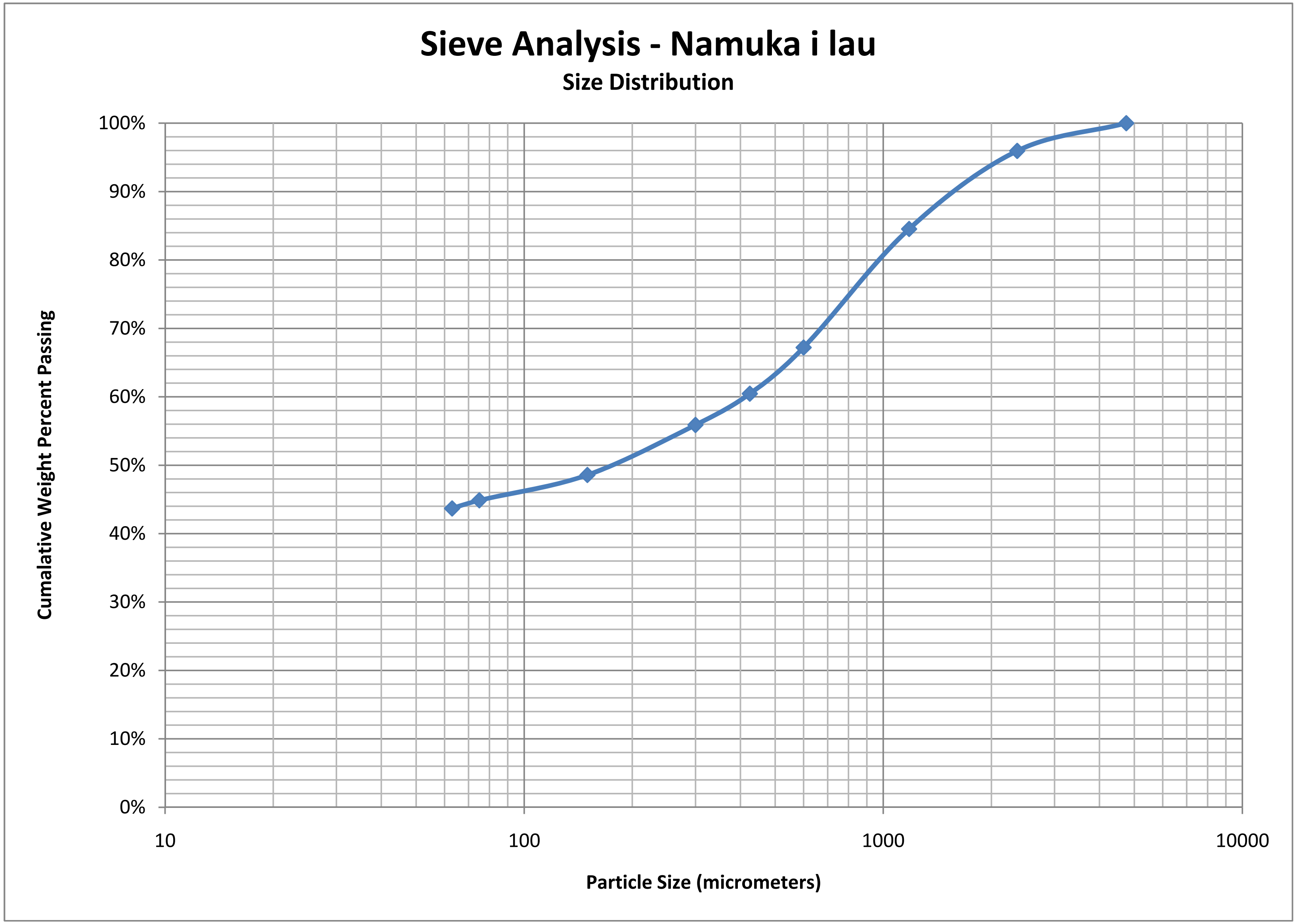


Figure 4.2 Particle size distribution (semi log plot) for Namuka I lau sample, excluding silt and clay fractions

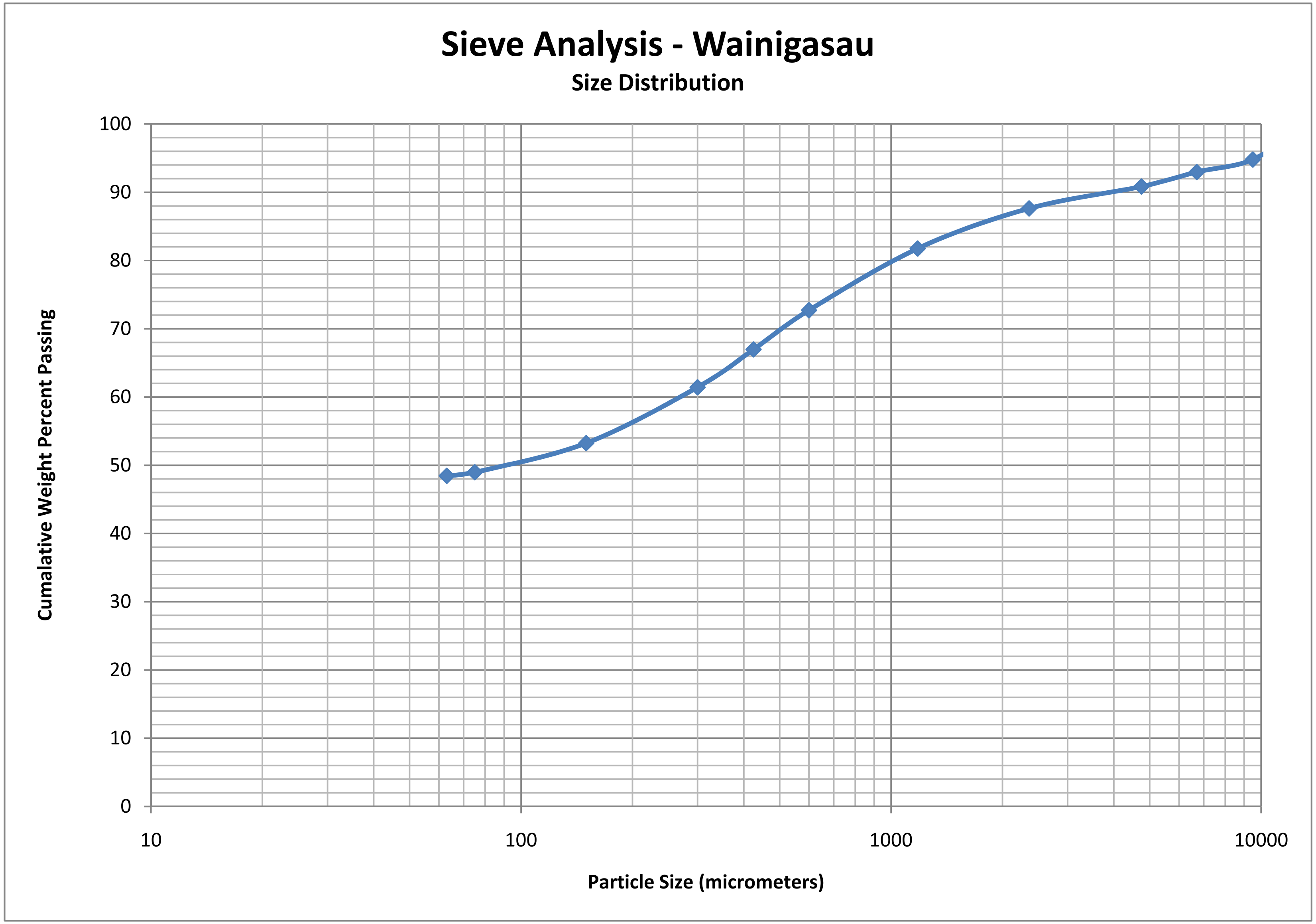


Figure 4.3 Particle size (semi log plot) distribution for Wainigasau sample, excluding silt and clay fractions

4.6 PERMEABILITY TESTING

4.6.1 Methodology

The main objective for carrying out permeability testing on the samples was to determine the coefficient of permeability or hydraulic conductivity, K, using the Falling Head Method. The samples tested from Namuka i lau and Wainigasau were sampled insitu and undisturbed samples were used to preserve as much as possible the internal structures of the material being tested.

A core cutter of equal diameter with the permeater cell was not available to procure the sample, so an alternative method of extracting the sample was devised by coring out the sample using the casing of the permeater cell, care being taken not to damage the permeater cell. During the sample preparation process, care was taken to ensure that there were no voids created in the sample through which water could have passed during the testing process and that good contact was maintained with the sides of the permeater; i.e. no air spaces were present between the sample and the permeater cell.

The sample was then trimmed flush (level) with the ends of the permeater before weighing of the sample and permeater was carried out. The permeater was placed in the cage and wire gauze was placed at the top of the sample before the top clamping plate was clamped evenly to the top of the permeater. The top inlet of the permeater cell was then connected to the glass T piece using rubber tubing and the branch of the T piece closed off. This was carried out in a bucket of water to assist de-airing. The assembled permeater cell was then placed in the immersion tank and filled slowing to prevent aeration of the water. The tank was filled to the overflow level.

The standpipe on the permeater cell was opened and the pinch clip released to allow full saturation of the sample to ensure no air was trapped in the apparatus. Considering the material tested had significant silt and clay fractions, the sample was left overnight to ensure complete saturation before testing began. Care was taken though to ensure that the water level did not fall below the minimum level. To begin the Falling Head Test, the pinch clip was closed, the standpipe to be measured filled to suitable levels and the other standpipe closed off.

The initial water level in the standpipe was recorded. The timer was started on release of the pinch clip and for the testing of our samples, the fall of the water level (h2) was recorded at the predetermined times of 1, 10, 100 and 1000 minutes. The permeater cell used was the WF26010 Falling Head Permeability Cell and the procedures stated above were based on the accompanying handbook.

The formula used to determine the coefficient of permeability, K, for the material being tested is

$$K = (a/A).(L/t).\ln\{ho/h\} \text{ or } K = 2.3(a/A).(L/t).\log_{10}\{ho/h\}$$

where K = hydraulic conductivity (m/s)

a = standpipe area (m2)

A – sample cross sectional area (m2)

L = sample thickness or height (m)

t = elapsed time from ho to h

ho = initial head (m) above top of sample

h = final head (m) at end of test or intermediate stage

4.6.2 Results

The hydraulic conductivity, K, values determined for the samples tested are presented in Table 4.7 below and the full calculations found in Appendix A.5:

Site	Hydraulic Conductivity, K (m/s)
Namuka i lau	2.78×10^{-8}
Wainigasau	9.35×10^{-7}

Table 4.7 Summary of results for hydraulic conductivity for Namuka i lau and Wainigasau samples

4.6.3 Discussion

The K values are within the range of clays as stated in Table 4.8 below:

Material	Hydraulic Conductivity (m/s)
Clay	$10^{-11} – 10^{-8}$
Silt, sandy silts, clayey sands, till	$10^{-8} – 10^{-6}$
Silty sands, fine sands	$10^{-7} – 10^{-5}$
Well sorted sands, glacial outwash	$10^{-5} – 10^{-3}$
Well sorted gravel	$10^{-4} - 10^{-2}$

Table 4.8 Typical values for hydraulic conductivities

4.7 SHEAR TESTING

4.7.1 Methodology

When dealing with slope stability, the material properties which are considered the most relevant are the angle of friction, the unit weight and the cohesive strength of the rock and soil material.

Friction and cohesion are best defined in terms of the Mohr-Coulomb plot of shear stress versus normal stress. The shear stress, τ, required to cause sliding increases with increasing normal stress, σ. The slope of the line relating shear stress to normal stress defines the angle of friction, Ø. If the surface is initially cemented or rough, a finite value of shear stress, τ, will be required to cause sliding when the normal stress level is zero. This initial value of shear strength defines the cohesive strength, c, of the surface.

The relationship between shear and normal stresses for a typical rock surface or for soil samples can be expressed as:

$$\tau = c + \sigma \tan Ø$$

(Hoek and Bray, 1981)

The strength of a soil can only be provided by the resistance to shearing of the soil structure. In terms of effective stresses, soils would be expected to obey a frictional failure criterion of the form:

$$\tau = c' + \sigma' \tan Ø'$$

where Ø' is the **effective angle of friction** or the **angle of shearing resistance.** The prime (')is added to the angle of friction (Ø') to indicate that it is a parameter that is related to effective stresses (Powrie, 2004).

Shear testing was carried out with the use of ring shear test apparatus (to determine the residual strength of the tested material) and was conducted on the matrix material of highly weathered conglomerates from the cut batter. Testing for residual strength was carried out as multiple failures had occurred at both the Namuka i lau and Wainigasau sites.

4.7.2 Description of Materials

The conglomerate matrix is described as a light brown to reddish, friable, highly weathered to completely weathered, silty clay (MH) with an invariably high plasticity index. The material has been exposed to past stress events as noted with past failure events and the presence of microfaulting and jointing related to the major Bay of Islands fault structures. The material is highly weathered and the reddish to orange brown colour is due to an abundance of hematite in the material, as well as the development of kaolinite clays.

4.7.3 Results

The results for the ring shear testing carried out on the polymict conglomerate matrix are summarised in the table below:

Mass on Hanger (kg)	Normal Stress (kPa)	Peak Shear Stress (kPa)
1	27.27	11.17
2	51.74	18.02
3	76.21	24.27
4	100.69	31.01

Table 4.9 Results of ring-shear test data for polymict conglomerate matrix

The results obtained were then plotted using the software RocPlot to plot normal stress versus shear stress (see Figure 4.4) to determine the residual friction angle, $Ø_r$, and residual cohesion, c_r.

The values of residual friction angle, $Ø_r$, and residual cohesion, c_r, as determined by plotting shear stress, τ, versus normal stress, σ, are:

Residual friction angle ($Ø_r$) = 15.0 degrees and

Residual cohesion (c_r) = 3.9 kilopascals (kPa)

4.7.4 Discussion

The ring shear test was conducted with the assumption that the surface sampled has undergone a considerable amount of strain and that $Ø_r$ is a basic requirement for stability. The Mohr-Coulomb diagrams (see Figure 4.4) show a friction angle, $Ø_r$, of 15.043 degrees and cohesion, c_r, of 3.92 kPa. Typical values for soil and rock properties from Hoek and Bray (1981) show the friction angle value of 15.043 as determined for the tested sample lies within the range of soft cohesive clays. The cohesion of 3.92 kPa as determined through the shear testing is low, comparably even lower than that of soft bentonite clays.

However, the above tests result may not represent in-situ conditions as coarse materials have been removed from the sample prior to ring-shear test. The tested sample therefore does not represent the true geological anisotropy and heterogeneity of the material tested, possibly influencing the outcome of the results. It is apparent that the tested conglomerate matrix has a very low cohesion value and a low internal friction angle, making it vulnerable to failure. Also, the high moisture content in the clay minerals is responsible for the reduction in effective stress and strength. Propagation of the failure surface through the polymict conglomerate occurs readily as the polymict conglomerate has been totally weathered to silt and clay fractions.

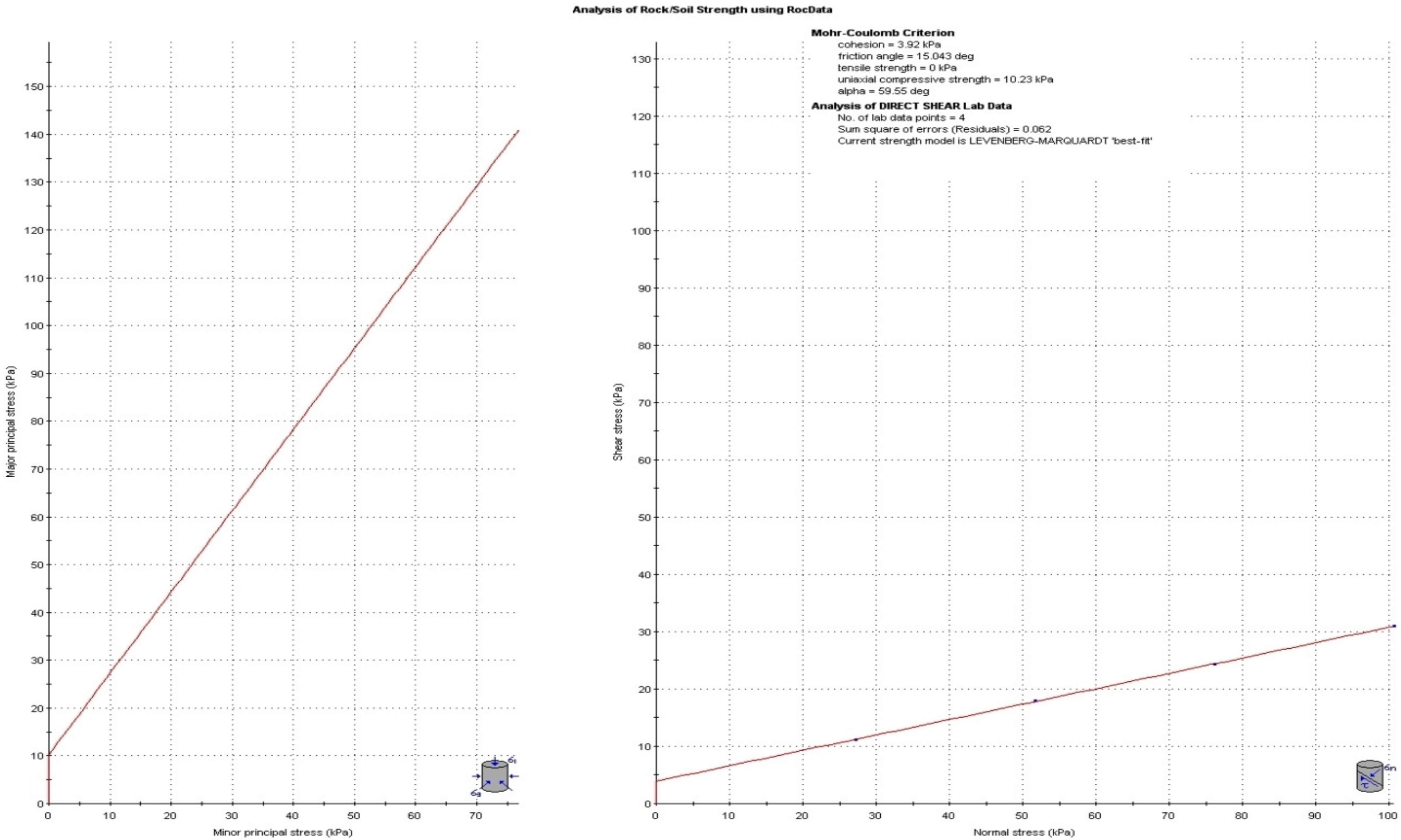


Figure 4.4 Mohr-coulomb diagrams for ring-shear tests carried out on Veisari Sandstone matrix material

4.8 SYNTHESIS

- XRD analysis determined the clasts and matrix material tested to be composed of varying compositions of quartz, kaolinite and hematite. Quartz compositions varied between 65% and 100%, kaolinite varied from <1% to 25% and hematite from <1% to 15%.

- Batter material tested had high water contents (Namuka i lau – 47.2%, Wainigasau – 44.9%) and unit weights of 14.5kN/m^3 and 15.8kN/m^3

- Atterburg Limits are summarised in the table below which show high liquid limits

	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
Namuka i lau	68	51	17
Wainigasau	76	61	15

- The results of the grainsize analysis are;

Sieve Size	Namuka i lau (%)	Wainigasau (%)
Gravel (> 4750 µm)	0	9.15
Sand (75 µm - 2360µm)	56.32	51.55
Silt and Clay (< 63 µm)	43.68	39.30

  and Namuka i lau is classified as a silty (clayey) sand and Wainigasau as silty gravely sand.

- A residual friction angle of 15º and a cohesion of 3.92 kPa were determined from the results of ring shear testing.

- Permeabilities are summarised in the table below and lie within the range of clays

Site	Hydraulic Conductivity, K (m/s)
Namuka i lau	2.78×10^{-8}
Wainigasau	9.35×10^{-7}

CHAPTER 5

GEOTECHNICAL MODELLING AND STABILITY ANALYSIS

5.1 INTRODUCTION

Fieldwork investigations and testing of slope forming materials in the laboratory allows investigation and understanding of the mechanisms and factors which contribute to the failure of slopes. The purpose of this chapter is to input the parameters determined through laboratory studies to produce a geotechnical model of the cut batters at Namuka i lau and Wainigasau to determine the factors of safety for the cut batters. Factors of safety are the accepted measure for slope stability.

A sensitivity analysis was also carried out by varying the geotechnical parameters of cohesion, c, and friction angle, γ, to analyse their effects on the failure mechanisms of the cut batters and their effects on the values of the factors of safety. Hydrogeological conditions are not presented in this geotechnical modelling as determined from the results of field investigations by the absence of a water table in the cut batters being modelled.

A back analysis was carried out using the geotechnical models created for Namuka i lau and Wainigasau to determine the reinforcement loads needed to achieve a factor of safety value of 1.5. These reinforcement load values were used to design appropriate stabilisation methods for the cut batters.

5.2 SLOPE STABILITY ANALYSIS USING SLIDE

5.2.1 Methods of Analysis

The factor of safety for any sloping surface is the ratio of the resisting forces, R, to the driving forces, D, represented by the formula:

$$FS = \frac{R}{D}$$

Factors of safety are used to classify slopes using the following criteria:

FS	>1.5	slope is stable and safe for building development
FS	>1	slope is stable
FS	=1	slope at limit equilibrium
FS	<1	slope is unstable

The Bishop Simplified and Janbu methods used in out geotechnical modelling are derivatives of the analysis method called the method of slices (see Figure 5.1). The method of slices procedure for analyzing stability in a slope is firstly by dividing the soil above the failure surface into a number of parallel vertical slices and the stability of each slice is calculated separately according to the formula:

$$FS = \frac{\sum(c' + \sigma' tan\emptyset')l}{\sum Wsin\alpha}$$

The Bishop Simplified method assumes that the tangential interslice forces are equal and opposite i.e.

$$X_1 = X_2$$

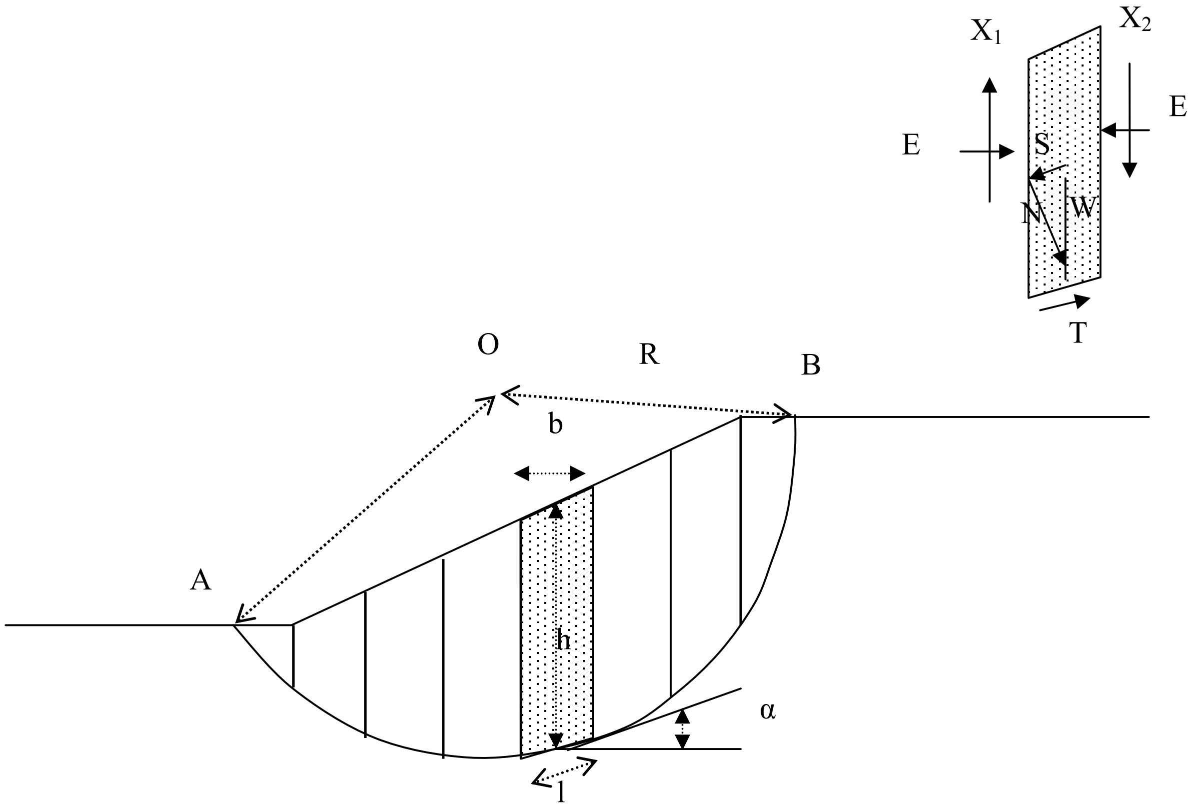


Figure 5.1 Dimensions and forces acting on a slope using the Method of Slices (adapted from Barnes, 1995)

but the normal interslice forces are not equal i.e.

$$E_1 = E_2$$

and the factor of safety is determined through the use of the formula:

$$FS = \frac{1}{\sum Wsin \propto} \sum \frac{[c'b + (W - ub)tan\emptyset]sec \propto}{1 + \frac{tan \propto tan\emptyset}{F}}$$

The Janbu method is used to analyse non-circular failure surfaces and is based on the assumption that the interslice forces are horizontal and there are no shear stresses between slices. The factor of safety is determined through the use of the formula:

$$F = \frac{\sum[c'b + (W + dX - ub)tan\emptyset'] \, m_{\propto}}{\sum(w + dX)tan \propto}$$

The SLIDE 5.0 software is a component of the Rocscience software suite which also includes other software's such as RocData, RocFall, RocLab, RocPlane, RocSupport, Dips, Examin, Swedge and Unwedge. The SLIDE software is two dimensional limit equilibrium slope stability software for soils and rock slopes. SLIDE has the ability to evaluate slope stability for both circular and noncircular failures from the simple to the complex models with external loading, groundwater and support. The SLIDE software carried out the analysis of the Wainigasau and Namuka I lau cut batters using the vertical slice limit equilibrium method, using both the Simplified Bishop Method and Janbu method to calculate factors of safety for the cut batters.

The models for the cut batters were created at a scale of 1: 500 from dimensions measured during the site investigations (see Table 3.1 and Table 3.2).

5.2.2 Namuka I lau Cut Batter Model

The Namuka I lau cut batter consists of 3 benches cut into highly weathered polymict volcanic conglomerate material. The batters are cut at approximately 50 degrees, with average bench widths of 1 metre and vertical heights of approximately 10 metres. The unit weight calculated and used in the modeling process is 14.5 kN/m^3 determined from laboratory testing and cohesion calculated as 3.92kPa with a friction angle of 15.0 degrees as determined from results of ring shear testing.

The factor of safety calculated using the Bishop Simplified method (see Figure 5.2) is:

$$FS = 0.425$$

and the factor of safety calculated using the Janbu Method is:

$$FS = 0.400$$

5.2.3 Wainigasau Cut Batter Model

The Wainigasau cut batter consists of 3 benches cut into highly weathered polymict volcanic conglomerate material. The benches are cut at approximately 60 degree slopes, with average bench widths of 3 metres and heights of approximately 10 metres. The unit weight calculated and used in the modeling process is 15.8 kN/m^3 and cohesion calculated as 3.92kPa with a friction angle of 15.0 degrees, all parameters being derived from laboratory testing.

The factor of safety calculated using the Bishop Simplified method (see Figure 5.3) is

$$FS = 0.438$$

and the factor of safety calculated using the Janbu Method is

$$FS = 0.420$$

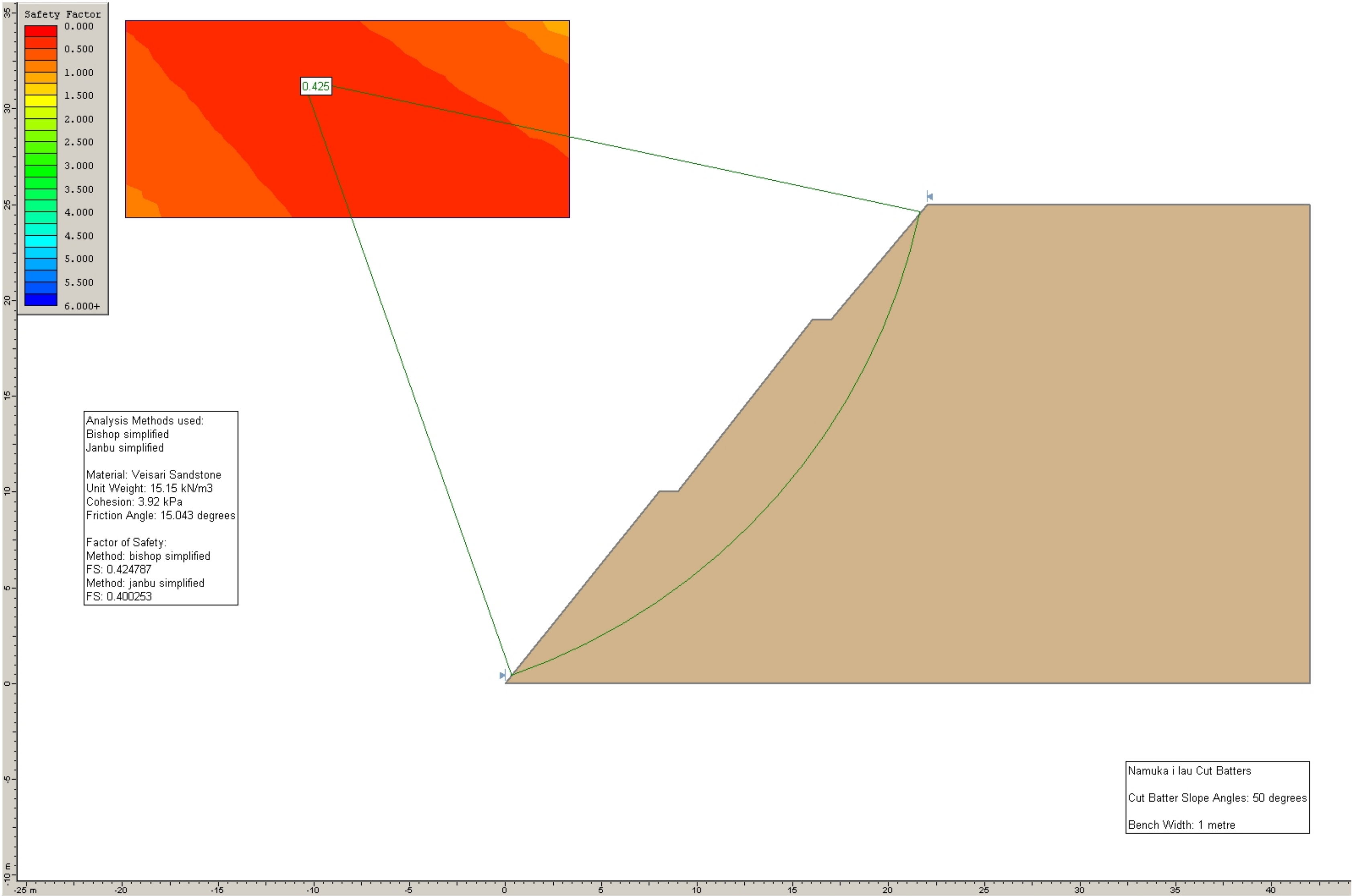


Figure 5.2 Slide analysis of Namuka I lau cut batter using Bishop Simplified Method

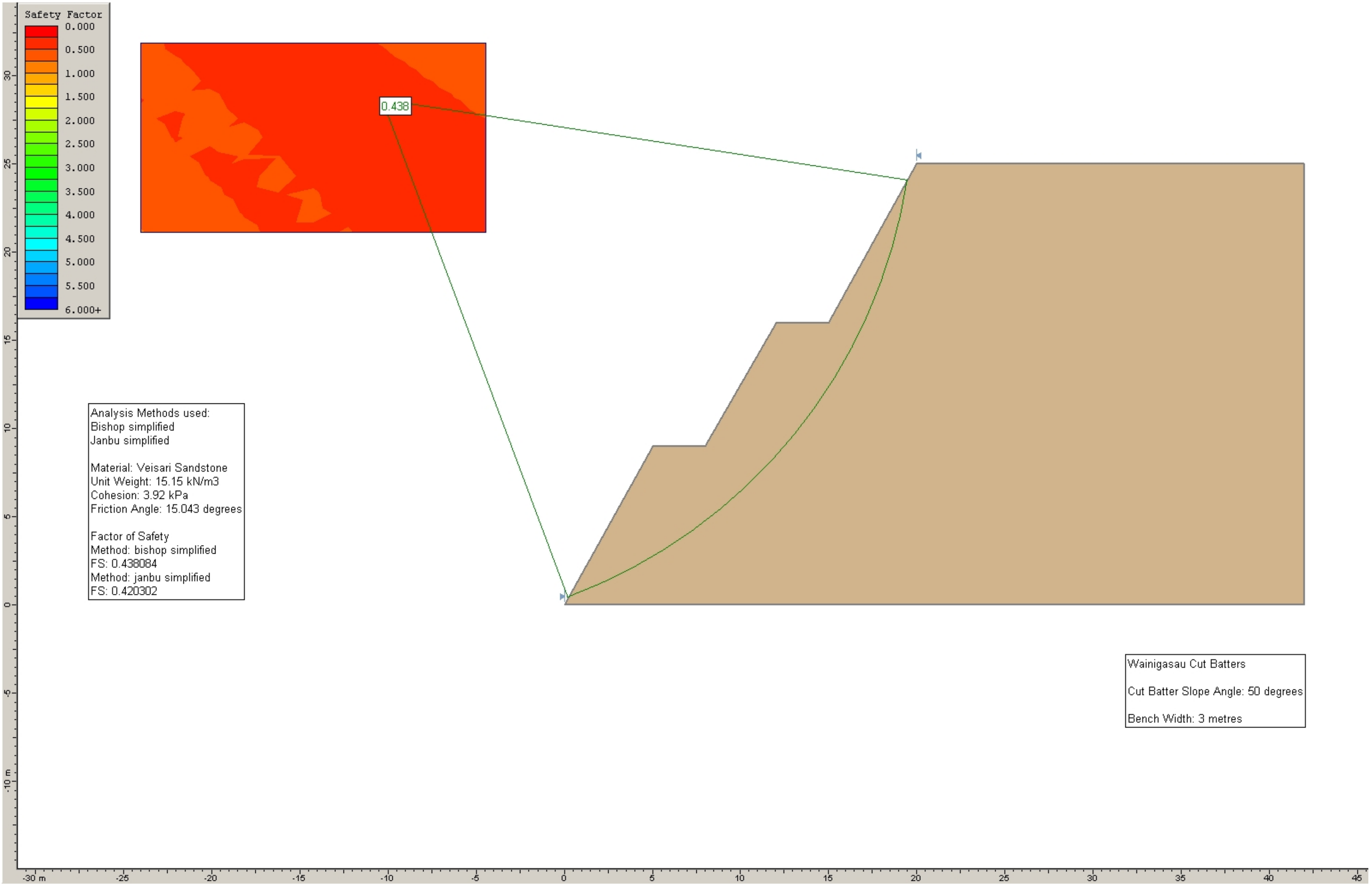


Figure 5.3 Slide analysis of Wainigasau cut batter using Bishop Simplified Method

5.2.4 Discussion

The results of the SLIDE modelling of the cut batters are summarised in Table 5.1 below.

Site Name	Method of Analysis	Factor of Safety (FS)
Namuka I lau	Bishop Simplified	0.425
	Janbu	0.400
Wainigasau	Bishop Simplified	0.438
	Janbu	0.420

Table 5.1 Summary results for SLIDE modelling for factors of safety

The results show that the factors of safety are well below the limit equilibrium value of 1.0. These low values for factor of safety using the SLIDE software can be assumed to be because of an underestimation of the friction angle and value of cohesion used in this modeling exercise. The residual strength and residual friction angles determined through ring shear testing of the finer fraction (<2mm) of the matrix of the polymict conglomerate will vary from the values of the material in the field due to:

- the nonhomogeneity of the material in the field – ring shear testing in carried out on finer fractions of the matrix which is not necessarily an accurate representation of the material in the field
- The finer material tested would consist mostly of clay material, which would have lower friction angles and residual shear strength values whereas the granular conglomerate has additional interlocking forces acting between the grains of matrix and clast material.

Other factors attributed to the underestimation of the values of cohesion and residual friction angles include:

- insufficient numbers of samples were available for testing, which would have provided a range of values for averaging

- a higher number of samples would also have provided a more representative spatial distribution of samples capturing the variability in material properties
- ring shear testing assumes previous shearing, and direct shear tests may have been more appropriate.

The cohesion determined through laboratory testing was significantly lower than typical values as summarized in Table 5.2 below. The cohesion calculated of 3.92 is significantly lower than 21 as calculated for materials of similar Atterburg Limits having a classification of MH (silt with high plasticity). The low cohesion value is calculated from residual strength values and the values in Table 5.2 represent values for peak drained strengths. This also applies to the disparity in the low residual friction angle calculated and the effective stress friction angle in Table 5.2.

Skempton (1975) in his fourth Rankine lecture alluded to this by saying that in the analysis of actual slips in clays, the values of the shear strength parameters as determined by conventional tests did not necessarily bear any relation to the values which must be operative in the clay at the time of failure. Therefore it is critical to try to understand why in certain cases there is a large discrepancy between ordinary laboratory test results and the actual field values of shear strength.

Unified classification	Relative compaction, RC(%)	Effective stress cohesion, c' (kPa)	Effective stress friction angle, Ø' (deg)
SM-SC	100	15	33
SC	100	12	31
ML	100	9	32
CL-ML	100	23	32
CL	100	14	28
MH	100	21	25
CH	100	12	19

Table 5.2 Typical Peak Drained Strengths for Compacted Cohesive Soils (U.S. Dept. Interior, 1973)

The factor of safety of 0.4 calculated, for Namuka i lau and Wainigasau, using the residual friction angle and cohesion, can be assumed to be the worst case scenario and field data collected verify this.

5.3 SENSITIVITY ANALYSIS

5.3.1 Introduction

The sensitivity analysis was carried out by varying the parameters cohesion, c, and friction angle, γ, to analyse their effects on the failure mechanisms of the cut batters and their effects on the values of the factors of safety. The cohesion and friction angle values used in this analysis represent average values for materials similar to those found at Namuka i lau and Wainigasau, but are not based on actual field or laboratory test results.

5.3.2 Methodology

The Bishop Simplified and Janbu methods were used in this analysis. The analysis was carried out by varying each of the parameters independently. The values used for cohesion were 10, 20, 25 and 30 kPa, and for friction angles were 10, 20, 25 and 30 degrees. The value of the unit weight was averaged between the two sites of Namuka i lau and Wainigasau with a value of 15.15 kN/m^3, which was assumed constant through the analysis.

5.3.3 Results

The results for the sensitivity analysis are summarised in Table 5.3 below:

Namuka ilau											
Properties	1	2	3	4	5	6	7	8	9	10	11
Gamma γ (kn/m3)	15.15	15.15	15.15	15.15	15.15	15.15	15.15	15.15	15.15	15.15	15.15
Phi (°)	15.043	15.043	30	30	25	15.043	20	15.043	20	25	30
Cohesion c' (kPa)	3.92	10	10	20	20	20	20	30	3.92	3.92	3.92
Factor of Safety											
Bishop Simplified	0.425	0.593	0.959	1.234	1.091	0.84	0.962	1.091	0.529	0.64	0.756
Janbu	0.4	0.482	0.91	1.209	1.089	0.895	0.987	invalid	0.499	0.602	0.713
Wainigasau											
Properties	1	2	3	4	5	6	7	8	9	10	11
Gamma γ (kn/m3)	15.15	15.15	15.15	15.15	15.15	15.15	15.15	15.15	15.15	15.15	15.15
Phi (°)	15.043	15.043	30	30	25	15.043	20	15.043	20	25	30
Cohesion c' (kPa)	3.92	10	10	20	20	20	20	30	3.92	3.92	3.92
Factor of Safety											
Bishop Simplified	0.438	0.617	0.989	1.28	1.141	0.891	1.01	1.168	0.539	0.644	0.748
Janbu	0.42	0.631	0.958	1.3	1.188	0.99	1.085	1.355	0.518	0.619	0.727

Table 5.3 Sensitivity results for Namuka i lau and Wainigasau

The resulting SLIDE figures can be found in Appendix A.7.

5.3.4 Discussion

The highest factors of safety were achieved when both the values of cohesion and friction angle were increased. The factor of safety for Namuka i lau and Wainigasau ranged from 0.8 to 0.9 for a cohesion value of 20 kPa but they achieved a value of 1 when the friction angles were increased at the same time. A factor of safety of 1.1 to 1.2 was achieved for a cohesion of 30kPa. The highest values for factor of safety were achieved when cohesion was 20 kPa and a friction angle 30 degrees. Increasing cohesion increases the factor of safety significantly as compared to an increasing friction angle which has a slightly lesser effect on the factor of safety. The factor of safety increases dramatically when both the values of cohesion and friction angle are increased.

The modelled higher factors of safety values represent a close representation to what is observed in the field at Namuka i lau and Wainigasau. The main trend developed during the sensitivity analysis is that cohesion within the batter material is the most significant factor for slope stability. A factor of safety value of 0.4 calculated initially for the cut batters cannot be justified as the cut batters currently remain stable.

The sensitivity analyses show to some extent the difference in design between the two sites. The Wainigasau site has higher factor of safety values than the Namuka i lau site and this could be attributed to the stability of 3 m wide benches as compared to 1 m wide benches at Namuka i lau.

We can assume that the low cohesion of 3.92 kPa determined through ring shear tests is an underestimation of the actual value of cohesion within the polymict conglomerates. The interparticle interactions between clay, clasts and matrix material would significantly increase the value of cohesion, possibly closer to the values of 20 kPa and 30 kPa which produced factors of safety equal to and greater than 1.0.

5.4 BACK ANALYSIS

5.4.1 Introduction

A back analysis was conducted using SLIDE to calculate the value of force required to reinforce the cut batter at the toe to increase the value of the factor of safety to a value of 1.5. The results of the back analysis would serve as a guide in the design of reinforcement structures.

The values determined through this back analysis would contribute in the design parameters of structures needed to reinforce the cut batters back to achieve acceptable factor of safety values;

$$FS \geq 1.5$$

Both active and passive force values were calculated in this back analysis. Active forces can be defined as those forces that act to reduce the driving force and passive forces are those that act to increase the forces of resistance.

Both methods of Bishop Simplified and Janbu were also used in this back analysis.

5.4.2 Results

The summary of results for the back analysis to achieve a factor of safety of 1.5 for the cut batters at Wainigasau and Namuka I lau (see Figure 5.4 and Figure 5.5) is summarized in the table below.

Site Name	Method of Analysis	Active Force (kN)	Passive Force (kN)
Namuka I lau	Bishop Simplified	741.904	1112.86
	Janbu	1279.13	1918.69
Wainigasau	Bishop Simplified	706.574	1059.86
	Janbu	1221.35	1832.02

Table 5.4 Summary results for SLIDE back analysis for calculation of reinforcement forces

5.4.3 Discussion

The passive forces calculated for the cut batters are the forces that are needed to increase the resisting forces which can be provided through retaining walls. The value of the active forces calculated is the amount needed to be removed from the driving forces to achieve a factor of safety of 1.5. This can be achieved at the Namuka i lau and Wainigasau sites through unloading by flattening of the slopes.

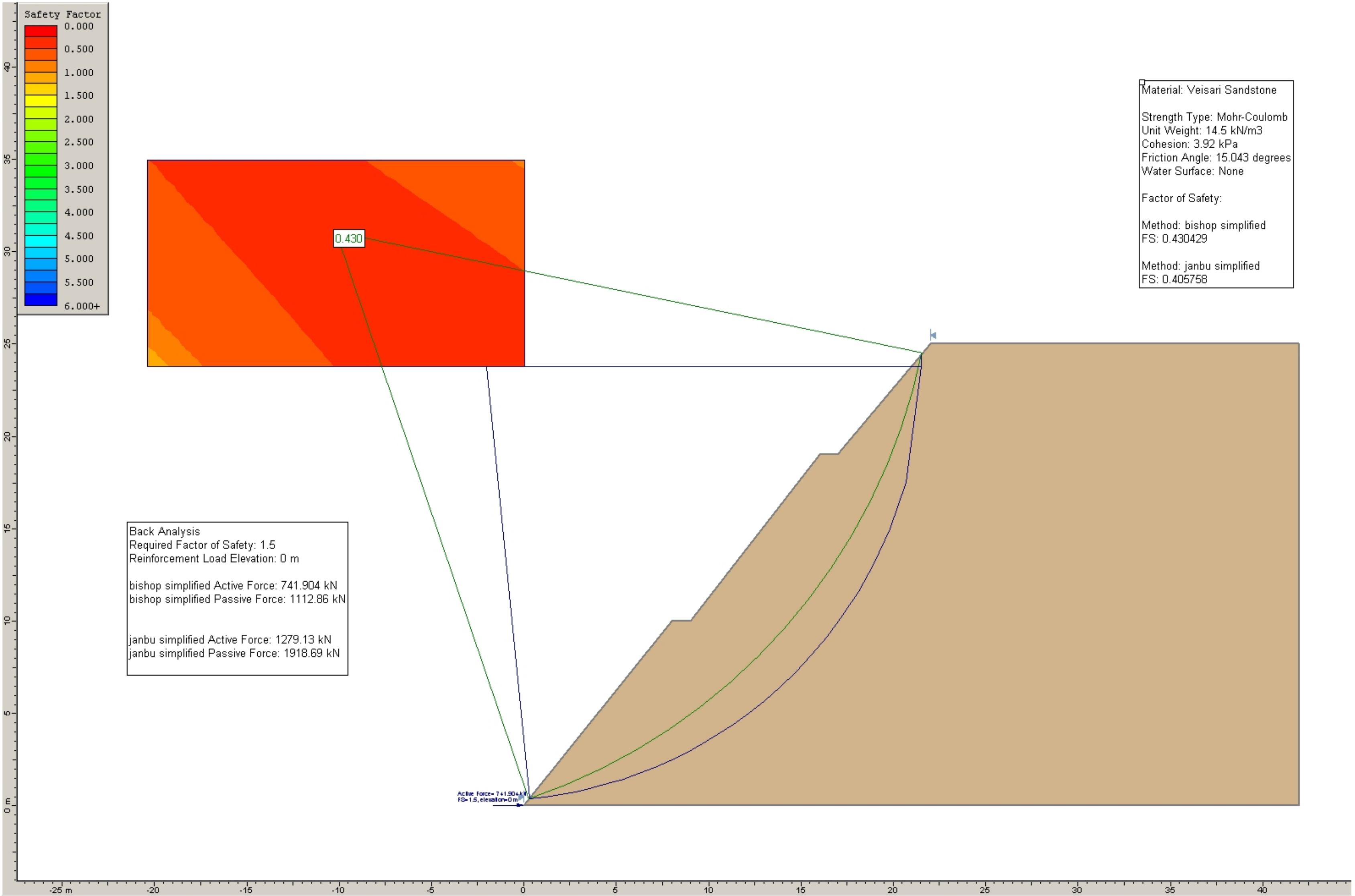


Figure 5.4 Back analysis using SLIDE for Namuka I lau cut batters

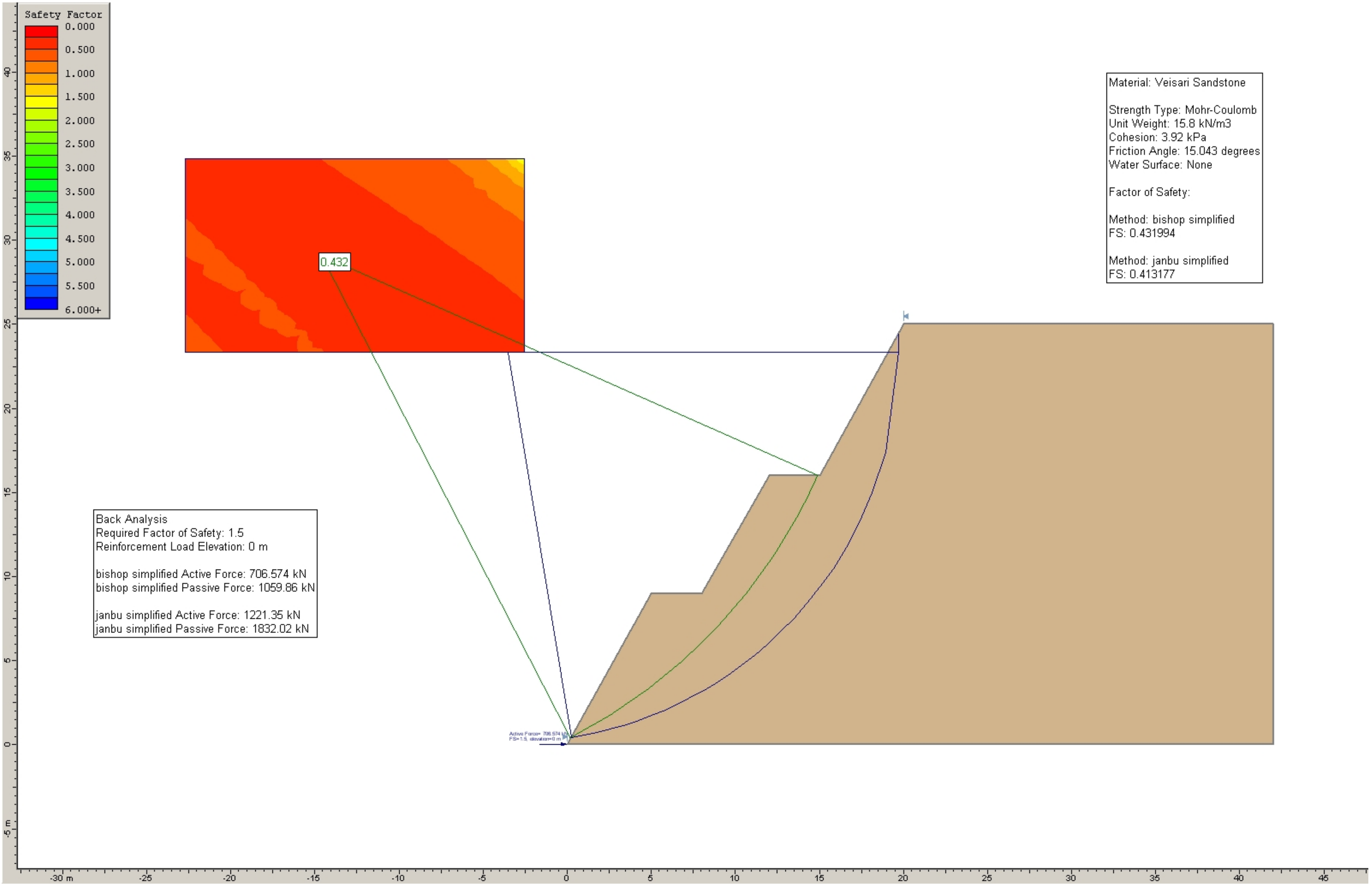


Figure 5.5 Back analysis using SLIDE for Wainigasau cut batters

5.5 REMEDIAL MEASURES

There are always two choices for improving a factor of safety:

(1) increase resistance or

(2) decrease driving forces or moments.

Measures to improve safety against rotational slope failure must increase the net resisting moment or decrease the net driving moment. Reduction of water forces by depressurization (drainage) is clearly beneficial. This action increases resisting moments and is the method of choice to improve the slope safety factor in wet ground.

Slope factor of safety is also increased by reducing weight.

Weight can be reduced in two ways:

(1) by decreasing the slope face angle and

(2) by removing material from the crest.

Another method of improving the safety factor is to buttress the toe. The effect of the toe buttress is that the buttress moment acting at the toe of the slope is added to the net resisting moment in the safety factor expression increasing the factor of safety (Pariseau, 2006).

There are several methods to slope stabilisation and they are namely unloading, buttressing, drainage, reinforcement, retaining walls, vegetation, surface slope protection and soil hardening.

5.5.1 Remedial Measures for Namuka I lau

Stabilisation of the cut batters at Namuka I lau is difficult because of the site location of the cut batters. Difficulties with this location are because of building developments that are situated adjacent to the crest of the cut batters and also that the cut batters are constructed adjacent to the major highway, Queens Road. This eliminates the possibility of unloading by decreasing the slope face angle of the benches as this would result in undercutting the foundations of the current building adjacent to the crest of the cut batters.

Stability methods recommended for the stabilisation of these cut batters would be the combination of the following methods of retaining wall, drainage and slope surface protection:

1. Retaining wall - Construction of a gabion retaining wall at the toe of the cut batters. This would have the similar effects to a rock buttress in that the weight of the gabion retaining wall at the toe of the slope would add to the resisting forces increasing the stability and overall factor of safety of the slope. The unit weight for basalt is between 17 and 22 kN/m3 (Hoek and Bray, 1981). Assuming that a 1 m x 1 m gabion was filled with basalt with a an average unit weight of 20 kN/m^3, the passive reinforcement force of approximately 1100 kN needed to stabilize the cut batters would be overcome with the use of 55, 1 m x 1 m gabion baskets. Rounding up to 60 baskets, effectively surpassing the calculated reinforcement forces by 100kN, with a 3, 2, 1 design this 3 m high gabion (see Figure 5.6) would provide sufficient reinforcement forces to stabilize the cut batter.

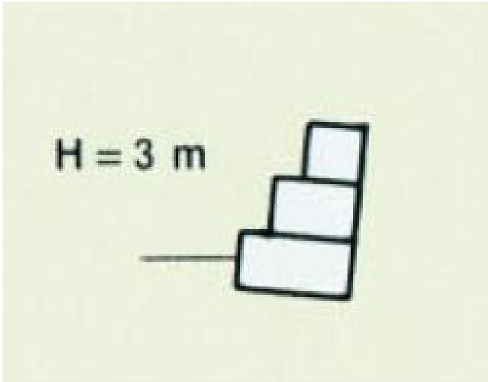


Figure 5.6 Gabion retaining wall design (3x2x1)

2. Drainage – Surface drainage is essential for the increased stability of the slope in that it reduces the effects of surface runoff over the slope reducing the destabilizing hydrostatic and seepage forces which contribute to the driving forces in slope stability. This new surface drainage system would include concrete interceptor drains along the crest of the cut batters, along benches and along the toe to capture all free draining surface runoff. This are in turn collected in downslope connector drains which discharge into the road stormwater drainage system. Also on longer cut batters, the concrete interceptor drains are connected to downslope collectors which discharge into the road stormwater drains.
3. Surface slope Protection – the main purpose for surface slope protection is to prevent infiltration by rainfall so that the slope can be maintained dry or partially dry. The recommended method would be the use of either shotcrete or chunam plaster. Chunam plaster would have the advantage or being slightly less costly as compared to the use of shotcrete (Abramson et al, 2002).

5.5.2 Remedial Measures for Wainigasau

The stabilisation of the cut batters at Wainigasau is relatively straightforward with no associated complications involving building developments, and sufficient room is also available adjacent to the highway for construction of stabilizing methods.

The stabilisation methods recommended for the Wainigasau cut batters would be a combination of methods including unloading, drainage and slope surface protection.

The recommendations for stabilisation of the Wainigasau cut batters are as follows:

1. Unloading – unloading is a technique that is used to reduce the driving forces within a slide mass. The most common methods of unloading are by excavation of the head of the slope, removing all unstable or potentially unstable materials, flattening slopes and benching. For the Wainigasau cut batters, the recommendations are that the material from the head of the cut batters be removed and that the slopes be flattened to recommended ratios for saprolite and residual soils. These are 1.5:1 and 2:1 respectively (Hunt, 2005).
2. Drainage – Surface drainage is essential for the increased stability of the slope in that it reduces the effects of surface runoff over the slope reducing the destabilizing hydrostatic and seepage forces which contribute to the driving forces in slope stability. The surface drains would consist of large concrete interceptor drains at the crest of the newly excavated head which would serve to reduce the amount of surface runoff infiltration over the cut batters: Concrete interceptor drains would also need to be installed at the base of each newly excavated cut batter to capture surface runoff of the cut batter faces and discharge them onto downslope concrete interceptor drains that collect the surface runoff from the horizontally placed drains and discharge them into the highway storm

drainage system. The damaged highway stormwater drainage system will also need to be repaired and checked to ensure that it will be able to handle the volume of surface runoff from the cut batters. Careful consideration is necessary to ensure that the stormwater drainage system is discharged appropriately into the natural drainage system of the area or out directly out to sea.

3. Surface slope Protection – the main purpose for surface slope protection is to prevent infiltration by rainfall so that the slope can be maintained dry or partially dry. The recommended method would be the use of either shotcrete or chunam plaster. Chunam plaster would have the advantage or being slightly less costly as compared to the use of shotcrete (Abramson et al, 2002).

5.6 SYNTHESIS

- The geotechnical model constructed using SLIDE version 5.0 calculated the relevant factors of safety using the Bishops Simplified and Janbu methods of analysis.

- The factors of safety calculated are summarised in the table below:

	Bishop Simplified	Janbu
Namuka i lau	0.425	0.400
Wainigasau	0.438	0.420

These values are too small and are attributed to the low values of cohesion and friction angle used in the modelling process and factors attributed to the results of the laboratory testing of the material

- Sensitivity analysis carried out by varying the values of cohesion and friction angle determined that cohesion is the major concerning factor slope stability and the value of cohesion would be greater in the field due to interparticle interactions between clay, clasts and matrix material.

- Back analysis determined the following reinforcement forces needed to achieve a factor of safety value of 1.5. The forces are summarised in the table below:

Site Name	Method of Analysis	Active Force (kN)	Passive Force (kN)
Namuka I lau	Bishop Simplified	741.904	1112.86
	Janbu	1279.13	1918.69
Wainigasau	Bishop Simplified	706.574	1059.86
	Janbu	1221.35	1832.02

- Stabilisation methods proposed for Namuka i lau are a gabion retaining wall (3x2x1x10) with a pyramid design with 3 gabions at base, 2 in the middle and 1 at the top, to provide the necessary passive force as calculated, drainage to control surface runoff and slope protection for the highly weathered polymict conglomerates. Stabilisation methods for Wainigasau are unloading through flattening of slopes, drainage and slope protection.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 THESIS OBJECTIVES

The focus of this thesis research was to investigate and understand cut batter failures that were occurring along the Queens Road in Fiji, specifically the Namuka i lau and Wainigasau sites.

The specific objectives for this thesis research were to:

- Prepare corridor maps (engineering geology and geomorphological) showing failure sites, types and relevant cross sections
- Develop an engineering geology and hydrogeological model for the cut batters that have failed
- Collect representative samples for clay mineralogy and other soil property tests
- Conduct stability analysis incorporating all relevant field and laboratory data to permit identification of remedial options and
- Recommend mitigation measures including but not limited to surface flows, seepage in bedrock and stable batter designs based on properties determined on rock and soil samples collected

6.2 GEOMORPHOLOGY

Namuka i lau and Wainigasau lie at the foot of coastal hills that extend north into the highlands of the Rama and Medrausucu ranges. The hilly and rolling landscapes of the southern coast transform northward into a rugged mountainous terrain with deeply incised creeks which drain into the Veisari and Wailekutu rivers.

6.2.1 Namuka i lau

The Namuka i lau site situated along the Queesn Road on the southern coast of Viti Levu is bordered to the west and north by coastal hills. The cut batters at the site were constructed during the construction of the Queens Highway which dissected a topographic knob to accommodate the new highway. Deposits of a landslide, reactivated during the construction of the cut batters have now been covered over by subdivision development except for hummocky ground at the crest of the current cut batters. Other events include debris slides occurring north of Namuka i Lau along the steep sides of deeply incised valleys of the higher mountainous Rama Ranges. The southern cut batter at the Namuka i lau has failed post construction and small failures are currently occurring in the northern cut batter.

The Namuka i lau site is drained to the north and south by creeks that drain out to the coast. Concrete v-drains at the foot of the cut batters discharge excess surface runoff to the highway stormwater system.

The Namuka i lau site is bounded to the west by the Baby of Islands Fault 1, to the east by the Bay of Islands Fault 2 and to the north by the Wailekutu fault but no structures were evident at the cut batter faces.

6.2.2 Wainigasau

The Wainigasau site is located at the base of coastal hills that extend southwards towards the coast from the higher Rama ranges. The cut batter has had multiple failures that has led to the reconstruction of the cut batters.

In contrast to the Namuka i lau site, the Wainigasau site has no evidence of failures of significant size in the vicinity of the cut batters both northwards (upslope) to the Rama ranges or laterally along the length of the but batters and adjacent coastal hills.

The surrounding coastal hills are drained through a network of creeks that drain northwards to the Veisari River and southwards to the coast.

The Wainigasau site is bounded to the north by the Wailekutu fault trending approximately east/west, and to the east by the Bay of Islands fault 1 and to the west by the Muaivuso fault.. Three faults were mapped on the face of the newly excavated cut batters and have orientations trending 356, 006 and 030, and having subvertical dips. Indications of normal movement on the fault are minor (< 5 centimetres).

6.3 GEOLOGY

The Namuka i lau and Wainigasau cut batters are constructed in the Veisari Sandstone which is made up of 2 members:

- a sandstone unit made up of fine sandstones, pebbly sandstones and a polymict conglomerate and
- the second member is a volcanic conglomerate member which is derived extensively from the andesitic Mau volcano.

The Namuka i lau and Wainigasau sites are both made up of the andesitic conglomerates of the Veisari Sandstone unit. The conglomerates belong to the first member which is derived from the Namosi andesites. The conglomerates are extremely weathered and iron oxidation has transformed its colour to a red – reddish brown colour. The conglomerates are matrix supported and largely polymict with majority of the clasts derived from the Namosi Andesites.

The conglomerates are extremely weathered and though clast and bedding forms may be seen in the excavated and exposed surfaces, upon sampling the conglomerate disintegrate into a fine to very fine clayey silt material.

Bedding forms in the units are sub horizontal to horizontal and strike orientations though difficult to measure because of the extreme weathering. Regional orientations of the unit strike east north east and have southerly dips of 5 to 15 degrees. There are indications of localised faulting and jointing exposed in the excavated faces of the cut batters. The microfaults measured at the Namuka i lau site have orientations from 348 to 022 with sub-vertical dips of 75 – 90 degrees.

6.4 GEOLOGICAL FIELD INVESTIGATIONS

6.4.1 Namuka i lau

The Namuka i lau site is made up of cut batters on either side of the Queens road. The cut batters were cut through a hill (~42 metres height at summit) during construction of the current Queens Road. The cut batters are aligned northeast/southwest with the northern cut batters sloping southeast and the southern cut batters sloping northwest. The cut batters cut on the Northern Side of the road have 3 benches cut at approximately 50 degrees. The northern cut batters are largely intact and undisturbed but small (< 2 metres width) failures have begun to occur on the lowest of the benches. Development has occurred above the cut batters post construction with housing construction being carried out post 1998 (last aerial photography) providing additional load forces. Groundwater is limited to surface runoff which flow freely over the cut batters and are collected in concrete v-drains which are connected to the highway storm water systems.

The cut batters on the southern side of the road have failed, post construction and all benches destroyed except for approximately 25 metres of bench on the western end of the cut batters. Two rows of gabion baskets have been constructed at the toe of the failed cut batter to act as a debris barrier in case of further failure. Similarly to the northern cut batters, groundwater is limited to surface runoff which flow freely over the cut batters and are collected in concrete v-drains which are connected to the highway storm water systems.

Both cut batters have sparse vegetation with benches being vegetated mainly with scrub, reeds and fern vegetation. Young tropical forest vegetation covers the top and surrounding slopes of the knob onto which the cut batters are constructed.

6.4.2 Wainigasau

The cut batter at Wainigasau is cut between the coastal hill ranges and the Waiqanake peninsula. The cut batter was originally cut as 3 benches (as determined from past aerial photos) but original slopes for the benches have been destroyed during failure of the cut batters beginning in 1996, 2003, 2007 and 2008. In May of 2008, after numerous failures, the cut batters were reconstructed into 3 new benches at slopes of 60 degrees.

The cut batters are cut in highly weathered conglomerates. The conglomerates are interbedded with mudstone and sandstone layers. The polymict conglomerate clasts are volcanic in origin and a high degree of weathering make original compositions difficult to determine. The conglomerate is matrix supported with matrix material consisting of medium to fine grained lithic material.

The cut batters are transacted by microfaults/fractures possibly related to the Muaivuso and Bay of Islands faults which lie to the east and west of the cut batters.

No springs or creeks are evident at along the cut batters but surface runoff have contributed significantly to surficial features of the cut batters and overall hydrological conditions of the cut batters. Surface rilling caused by excessive surface runoff is evident down the slopes of the three benches. Existing microfaults and fractures have been enlarged by surface runoff and these weakened structures contribute to the development of future failure planes.

Constructed drainages along the benches are sediment logged and blocked and concrete v-drains at the base of the cut batters have been destroyed by previous failures and during the reconstruction process.

The southern cut batter is stable and vegetated with young tropical vegetation with surface runoff being collected by a concrete v-drain at the foot of the cut batter.

6.5 LABORATORY STUDIES

Geotechnical testing carried out on the highly weathered polymict conglomerates sampled from the cut batters included composition analysis using x-ray diffraction analysis, grainsize analysis, determination of water content and bulk unit weight, classification of the material through the determination of Atterburg Limits including liquid limit, plastic limit and plasticity index, permeability testing and shear testing.

The material tested achieved the following properties:

The XRD analysis revealed the clasts and matrix were composed of varying percentages of quartz, kaolinite and hematite:

Quartz: 65% - 100%

Kaolinite: <1% - 25%

Hematite: ,1% - 15%

Grainsize Analysis carried out using sieve analysis achieved the following results for the two areas of Namuka i lau:

Gravel (> 4750 μm) 0%

Sand (75 μm - 2360μm) 56.32%

Silt and Clay (< 63 μm) 43.68% and

Wainigasau:

Gravel (> 4750 μm) 9.15%

Sand (75 μm - 2360μm) 51.55%

Silt and Clay (< 63 μm) 39.30%

Water content and bulk unit weights for the two areas are:

Namuka i lau: Water Content, $w = 47.18\%$

Bulk Unit Weight, $\gamma_b = 14.5$ kN/m^3 and

Wainigasau: Water Content, $w = 44.92\%$

Bulk Unit Weight, $\gamma_b = 15.8$ kN/m^3

Atterburg Limits determined for material from the two areas are:

Namuka i lau: Liquid Limit, LL = 68

Plastic Limit, PL = 51

Plasticity Index, PI = 17

Plasticity Chart Classification: MH and

Wainigasau: Liquid Limit, LL = 76

Plastic Limit, PL = 61

Plasticity Index, PI = 15

Plasticity Chart Classification: MV

Permeabilities for the two areas were determined by carrying out the Falling Head Test on samples from the two sites and the the hydraulic conductivity, K, values determined for the samples tested are:

Wainigasau: $K = 6.66 \times 10^{-7}$ m/s and

Namuka i lau: $K = 2.78 \times 10^{-8}$ m/s

Ring Shear testing was carried out on the matrix material of the polymict conglomerate to determine the residual friction angle and cohesion for the material. The values of friction angle, Ø, and cohesion, c, as determined by plotting shear stress, τ, versus normal stress, σ, are:

Friction angle (Ø) = 15.043 degrees and

Cohesion (c) = 3.92 kilopascals (kPa)

6.6 GEOTECHNICAL MODELLING

Geotechnical modelling was carried out using the SLIDE software to determine factors of safety for the cut batter slopes at Namuka i lau and Wainigasau. The factors of safety determined using the Simplified Bishop and Janbu mthods are as follows:

Namuka i lau:	Bishop Simplified method $FS = 0.432$	and
	Janbu Method;$FS = 0.406$	
Namuka i lau:	Bishop Simplified method $FS = 0.433$	and
	Janbu Method;$FS = 0.415$	

A sensitivity analysis carried out by varying the values of cohesion and friction angle achieved factors of safety greater than 1 when cohesion was increased to 30kPa and also when both the values of cohesion and friction angle were increased to 20kPa and 30º respectively.

A Back Analysis carried out to determine the force needed to reinforce the cut batters at the toe, to achieve a factor of safety of 1.5, calculating both active and passive forces yielded the following results:

Namuka I lau	Bishop Simplified	Active Force (kN) = 683.103
		Passive Force (kN) = 1024.65
	Janbu	Active Force (kN) = 1135.62
		Passive Force (kN) = 1703.43
Wainigasau	Bishop Simplified	Active Force (kN) = 653.022
		Passive Force (kN) = 979.533

Janbu	Active Force (kN) = 1068.74
	Passive Force (kN) = 1603.12

6.6.1 Remedial Measures

Remedial measures for Wainigsau include:

- construction of a gabion retaining wall using a 3x2x1 design to increase resisting forces, increasing the stability of the slope and the overall factor of safety
- Improving drainage by installing concrete drains and interceptor drains to capture surface runoff from the cut batter slopes and discharge them into the highway stormwater drain system
- Slope protection through the use of either shotcrete or chunam dependent on cost restrictions

Remedial measures for Wainigsau include:

- Unloading by the flattening of the cut batter slopes to ratio of 2:1
- Improving drainage by installing concrete drains and interceptor drains to capture surface runoff from the cut batter slopes and discharge them into the highway stormwater drains which also need replacing.
- Slope protection through the use of either shotcrete or chunam dependent on cost restrictions

6.7 CONCLUDING STATEMENT

Cut batter failures are a widespread phenomena affecting majority of roads in Fiji. Basic data and poor application of this data can be attributed to the poor design of many of these cut batters. A basic understanding of the factors affecting slope stability, the properties of the material into which the cut batters are constructed and basic rock mechanics allow us to make logical and accurate assumptions during the design and construction of these cut batters. These basic assumptions have the potential to drastically increase the long term stability of these cut batters which in turn effectively reduce the economic strain it places on rehabilitating the increasing occurrences of cut batter failures that affect Fiji's national road network.

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Appendix

Appendix

A.1 **XRD Analysis Plots**

A.2 **Determination of Bulk Unit Weight and Water Content**

A.2 **Atterburg Limits Datasheet**

A.3 **Grainsize Analysis Datasheet**

A.4 **Falling Head Data Calculations**

A.5 **Definitions: Factor of Safety and Methods of Analysis**

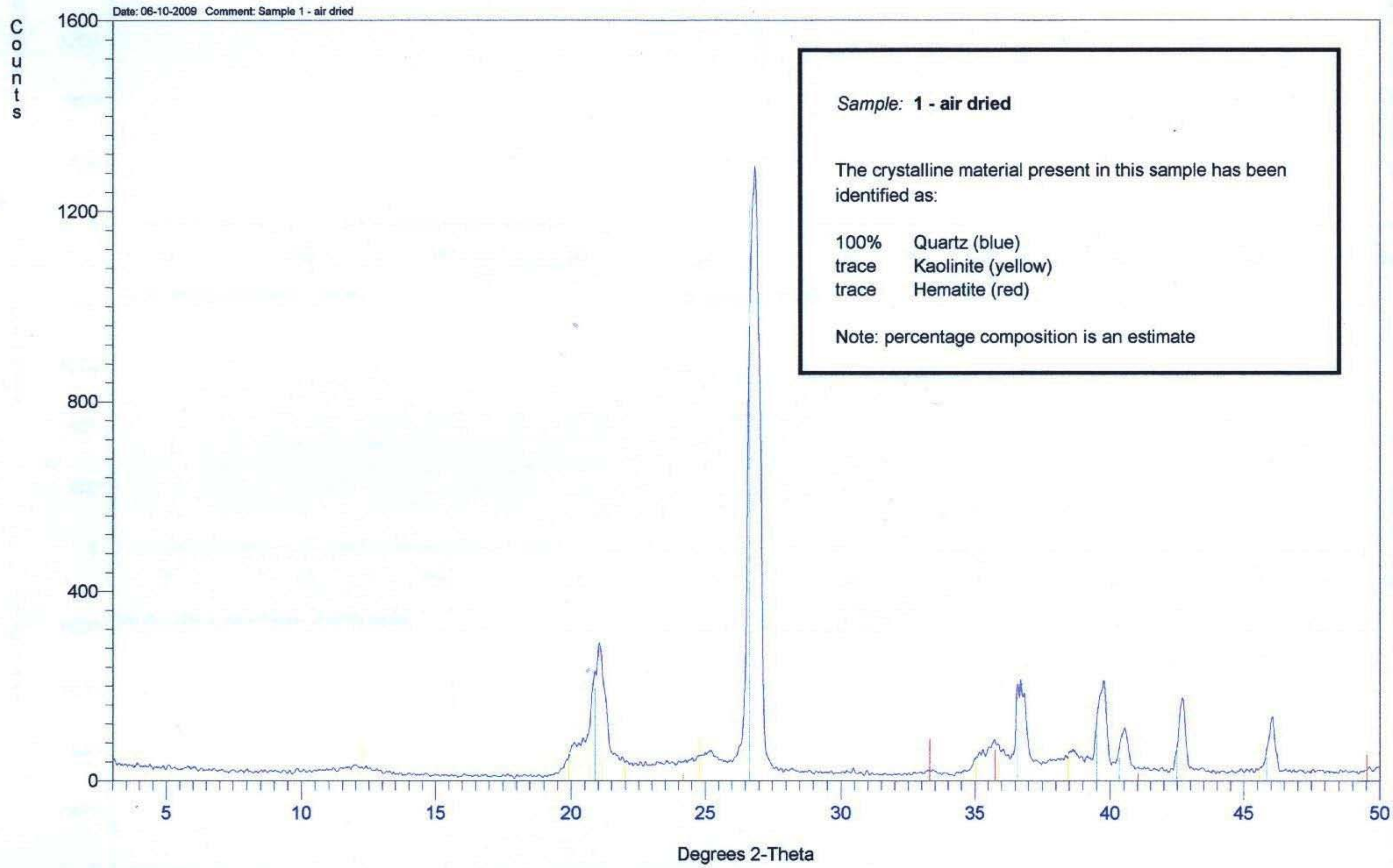
A.6 **Sensitivity Analysis: SLIDE computations for factor of safety**

Appendix A.1

XRD Analysis Plots

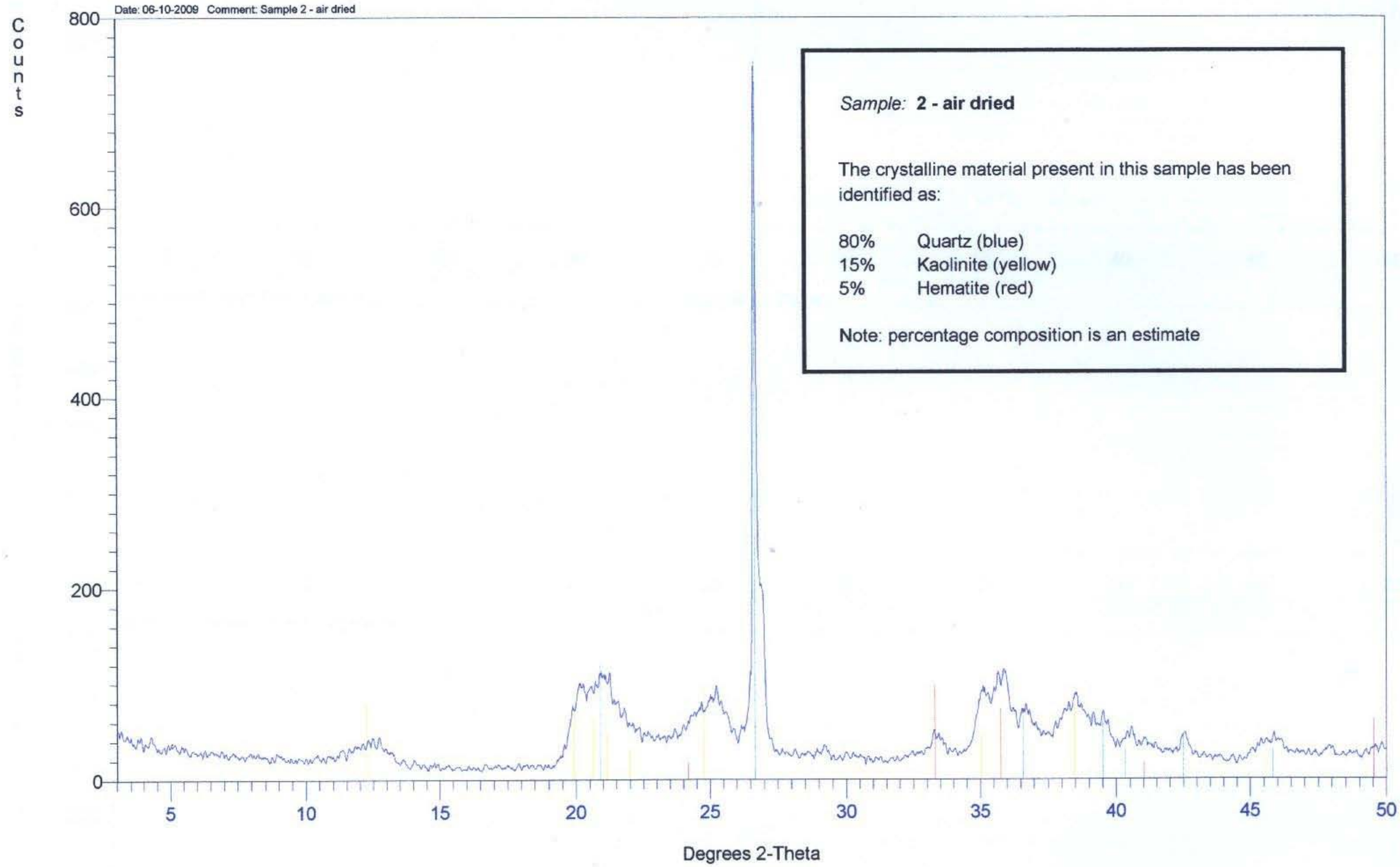
Geochemistry Laboratory, Department of Geological Sciences, University of Canterbury, New Zealand

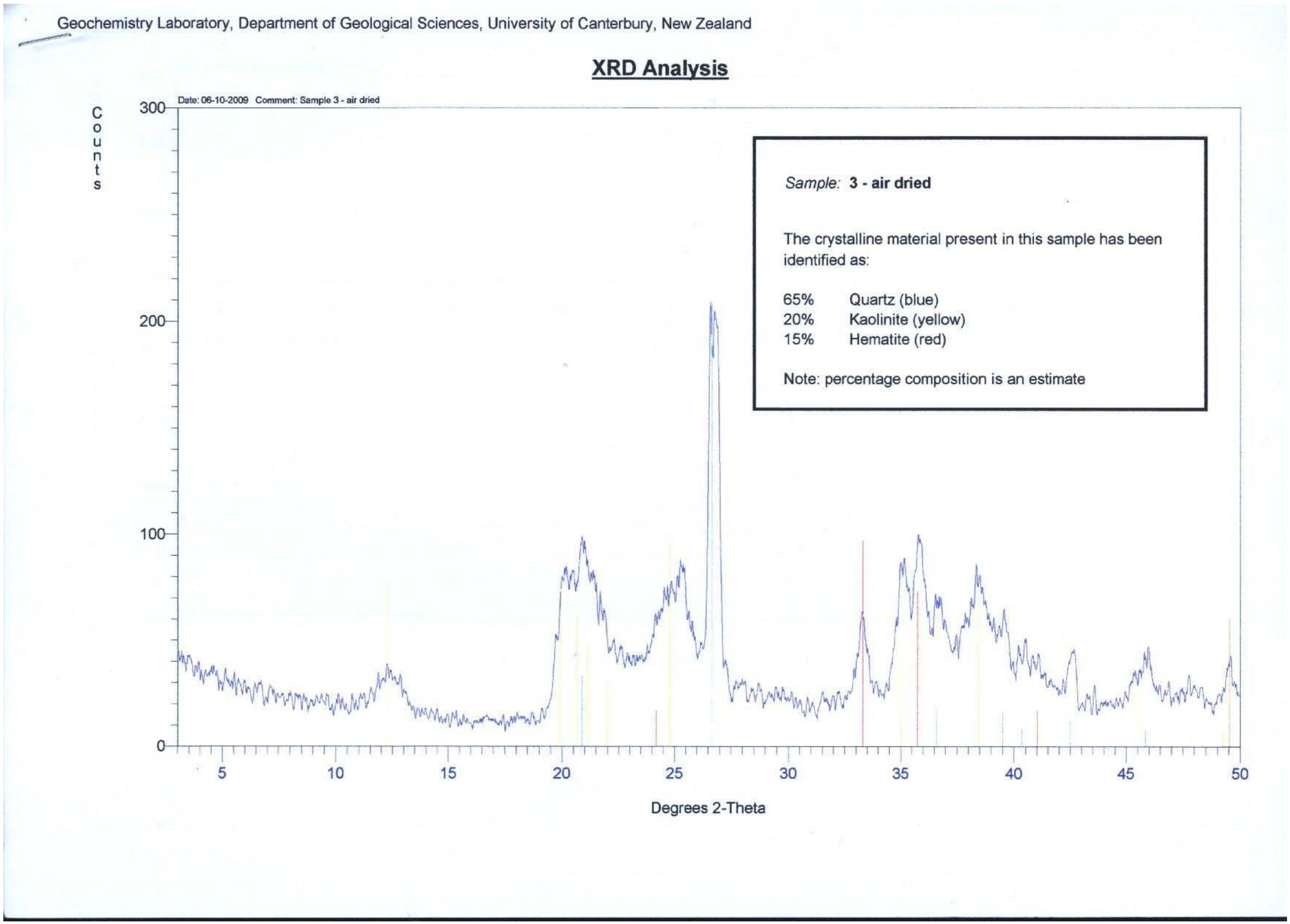
XRD Analysis



Geochemistry Laboratory, Department of Geological Sciences, University of Canterbury, New Zealand

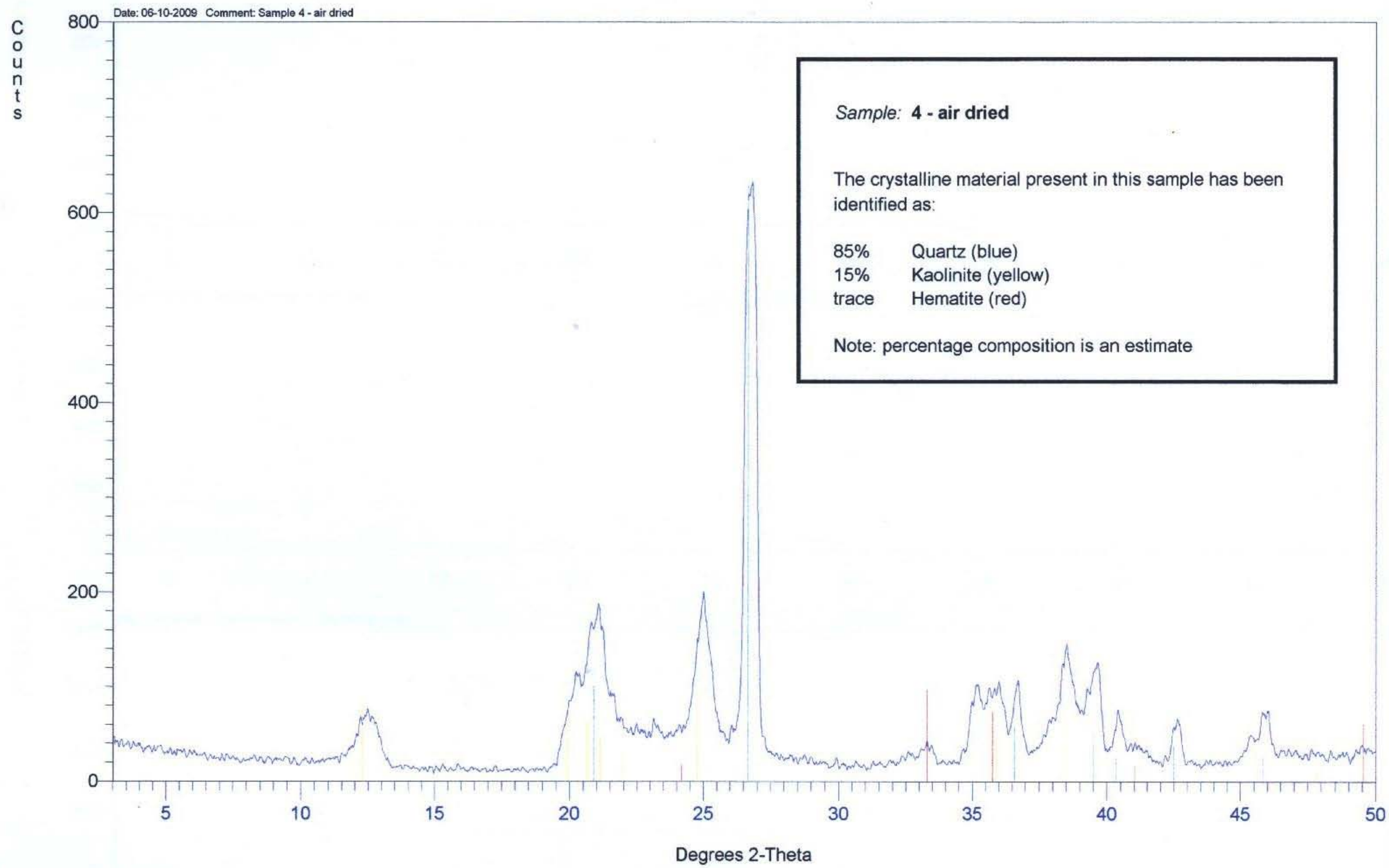
XRD Analysis




Geochemistry Laboratory, Department of Geological Sciences, University of Canterbury, New Zealand
XRD Analysis
Date: 06-10-2009 Comment: Sample 3 - air dried
Counts
300
200
100
0
5
10
15
20
25
30
35
40
45
50
Degrees 2-Theta
Sample: 3 - air dried
The crystalline material present in this sample has been identified as:
65% Quartz (blue)
20% Kaolinite (yellow)
15% Hematite (red)
Note: percentage composition is an estimate

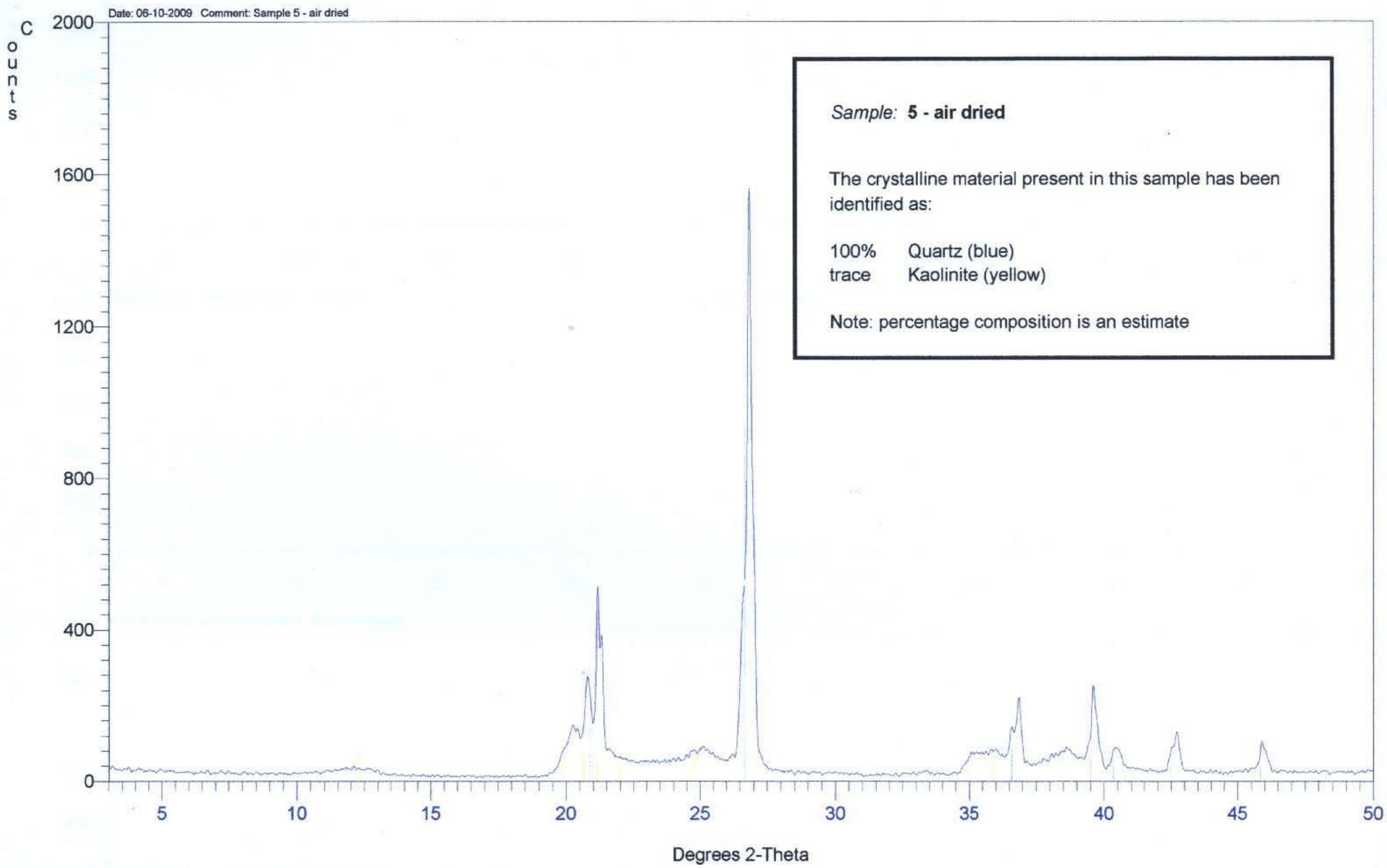
Geochemistry Laboratory, Department of Geological Sciences, University of Canterbury, New Zealand

XRD Analysis



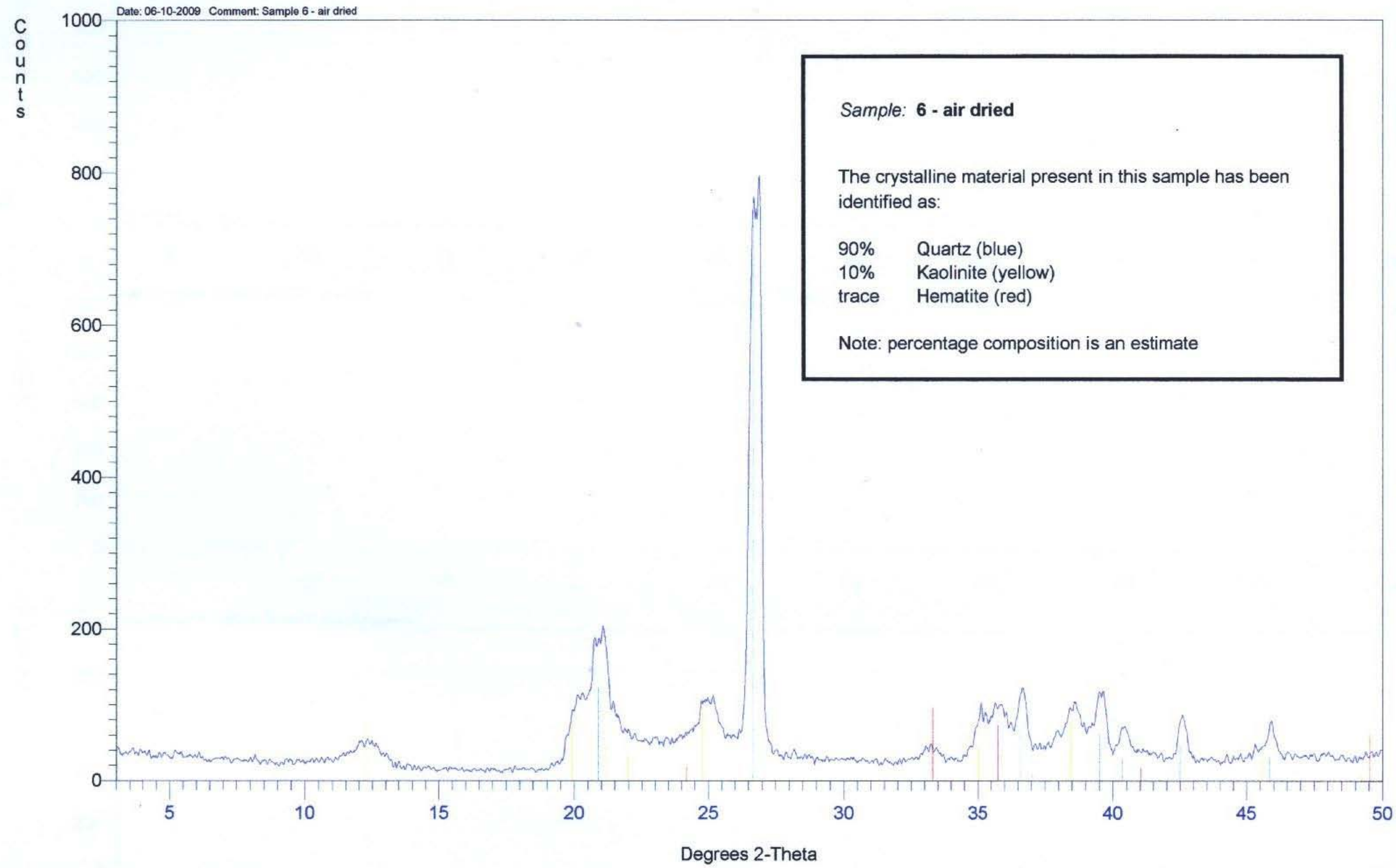
Geochemistry Laboratory, Department of Geological Sciences, University of Canterbury, New Zealand

XRD Analysis



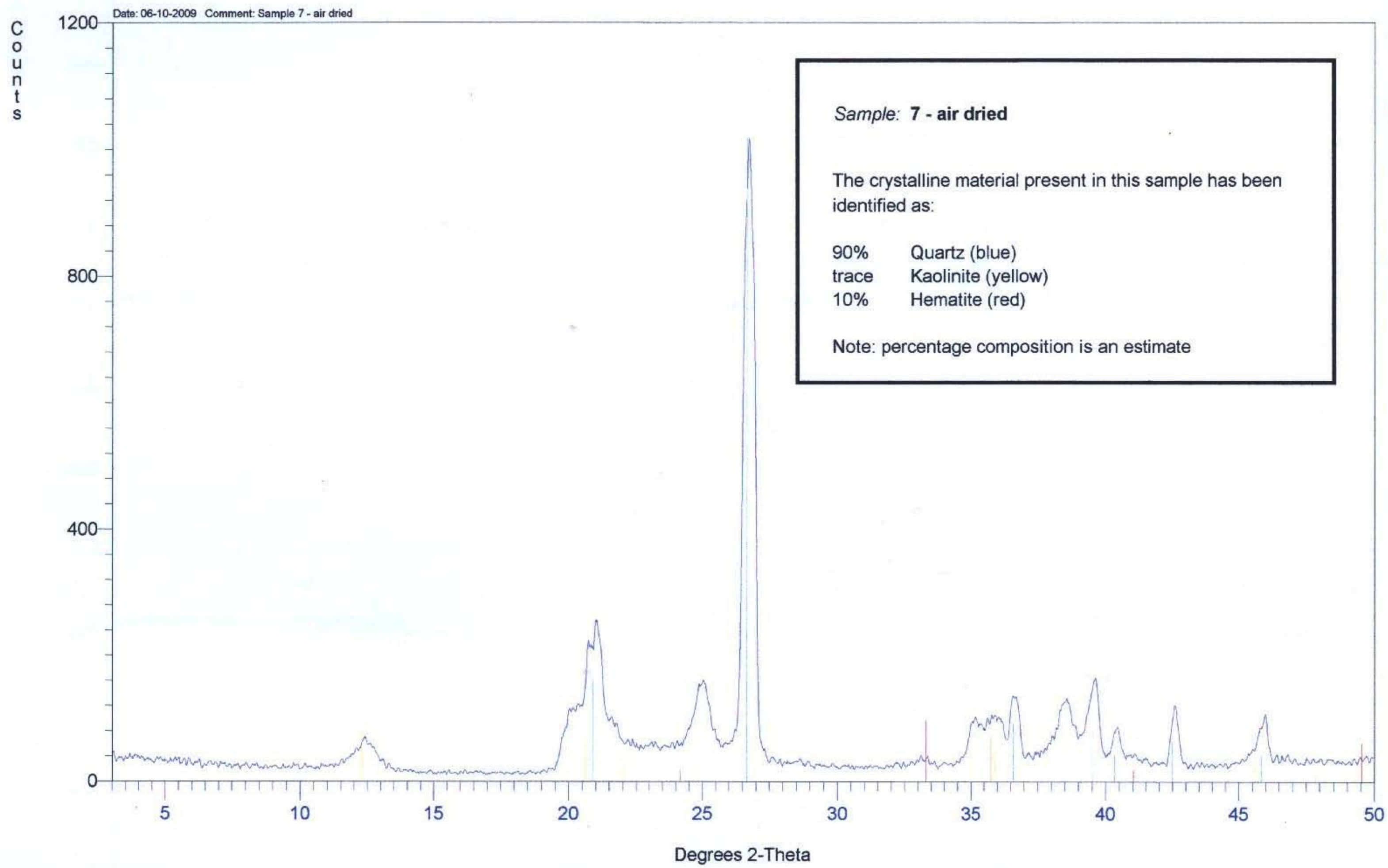
Geochemistry Laboratory, Department of Geological Sciences, University of Canterbury, New Zealand

XRD Analysis



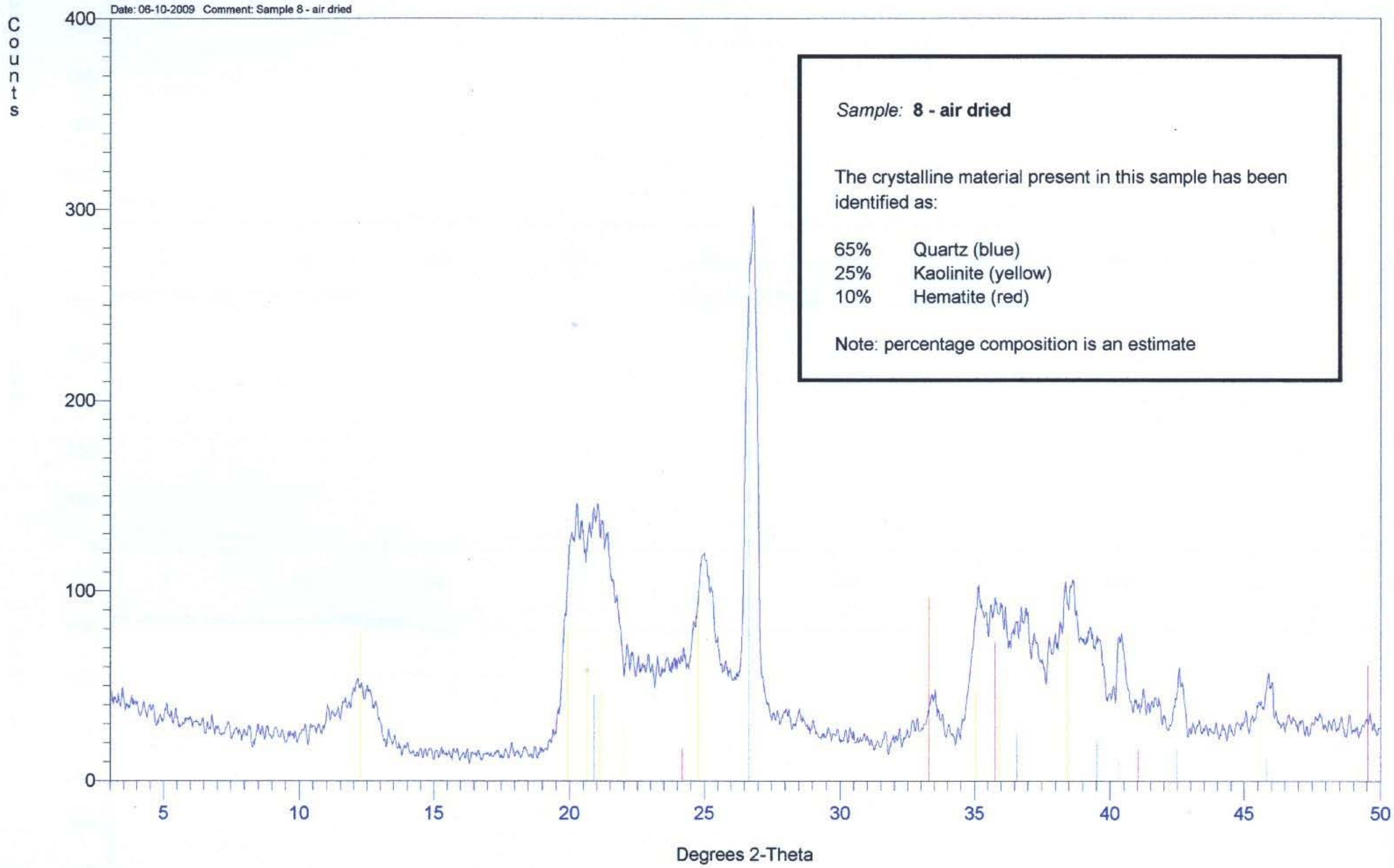
Geochemistry Laboratory, Department of Geological Sciences, University of Canterbury, New Zealand

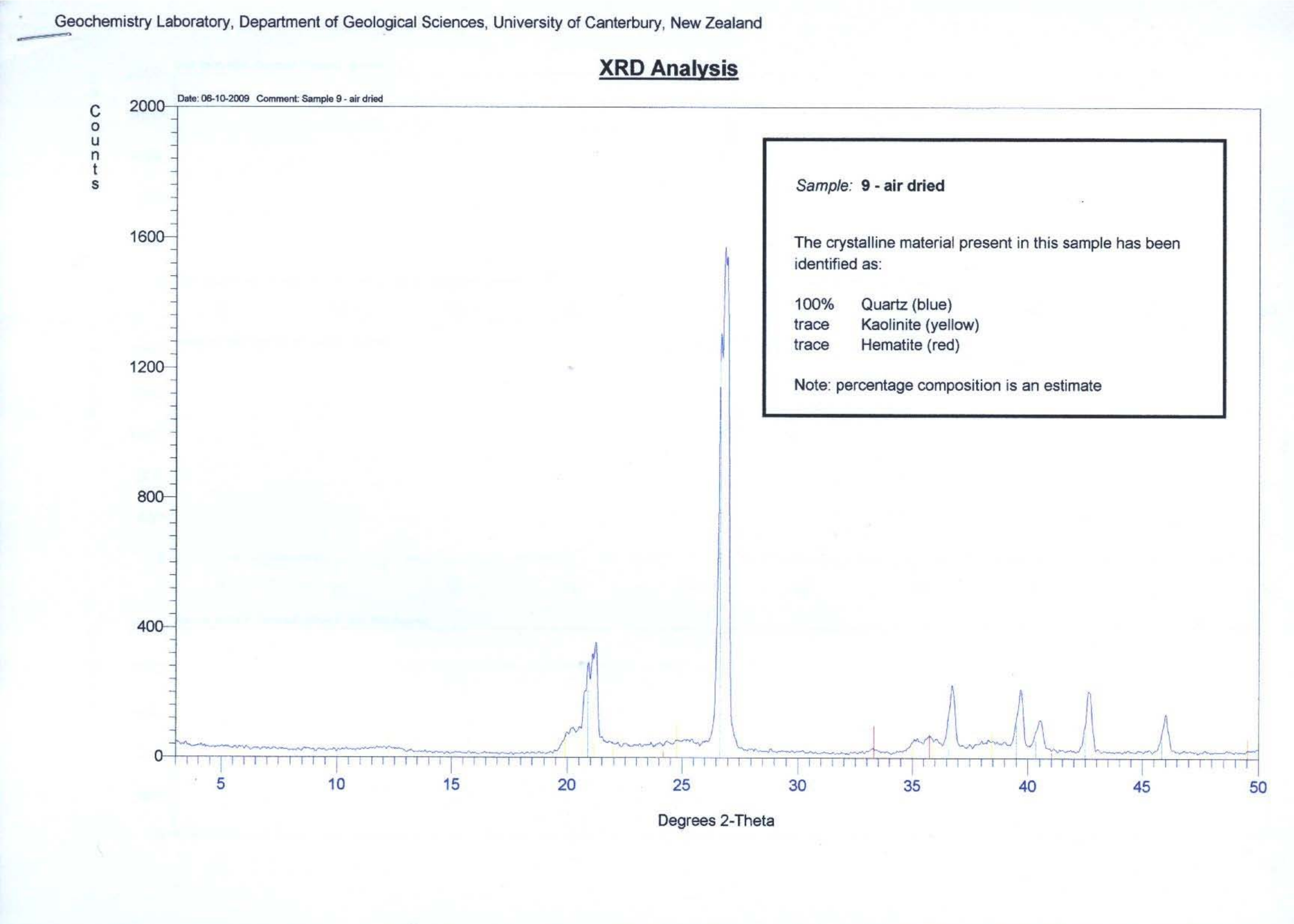
XRD Analysis



Geochemistry Laboratory, Department of Geological Sciences, University of Canterbury, New Zealand

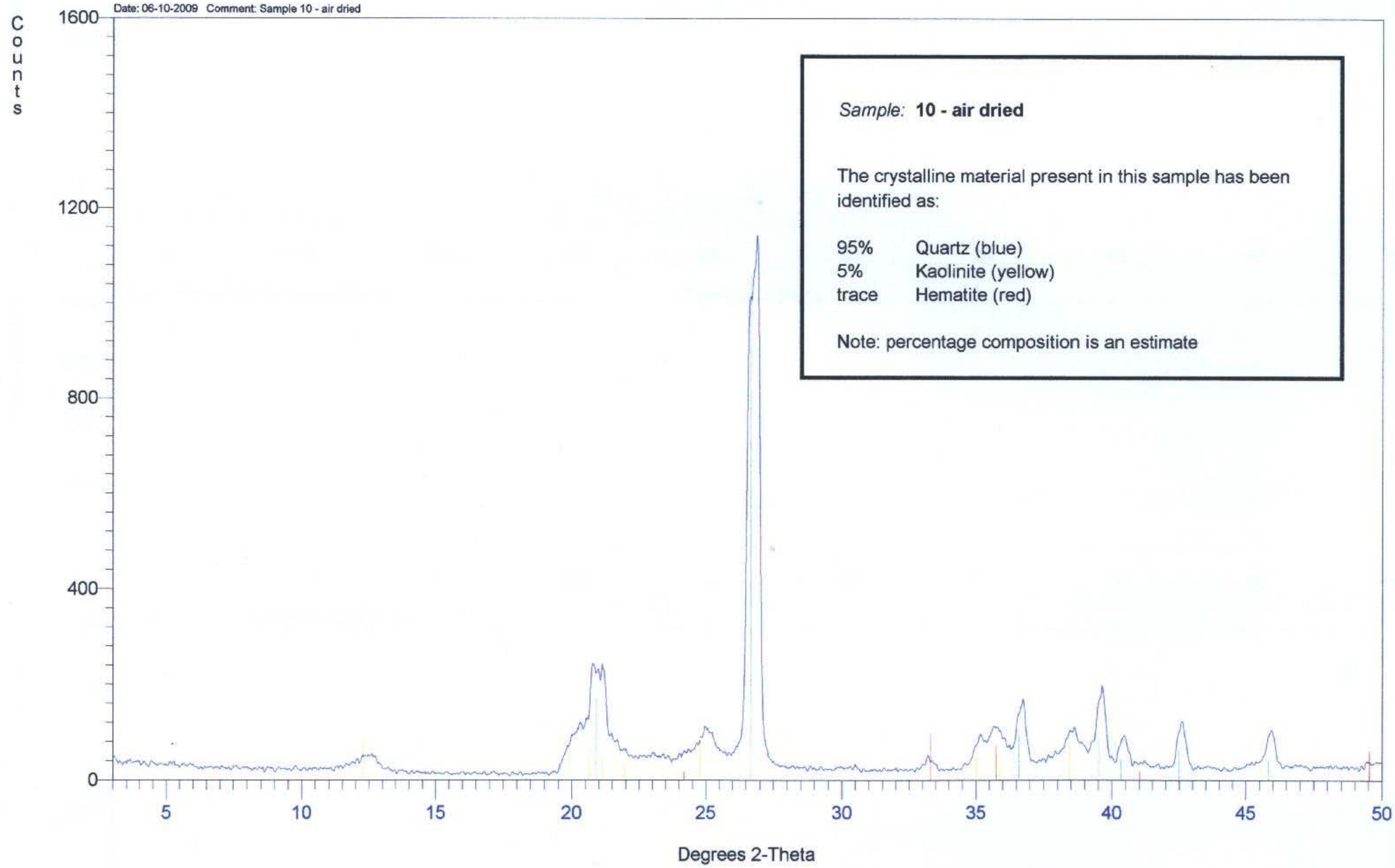
XRD Analysis




Geochemistry Laboratory, Department of Geological Sciences, University of Canterbury, New Zealand
XRD Analysis
Date: 06-10-2009 Comment: Sample 9 - air dried
Counts
2000
1600
1200
800
400
0
5
10
15
20
25
30
35
40
45
50
Degrees 2-Theta
Sample: 9 - air dried
The crystalline material present in this sample has been identified as:
100% Quartz (blue)
trace Kaolinite (yellow)
trace Hematite (red)
Note: percentage composition is an estimate

Geochemistry Laboratory, Department of Geological Sciences, University of Canterbury, New Zealand

XRD Analysis

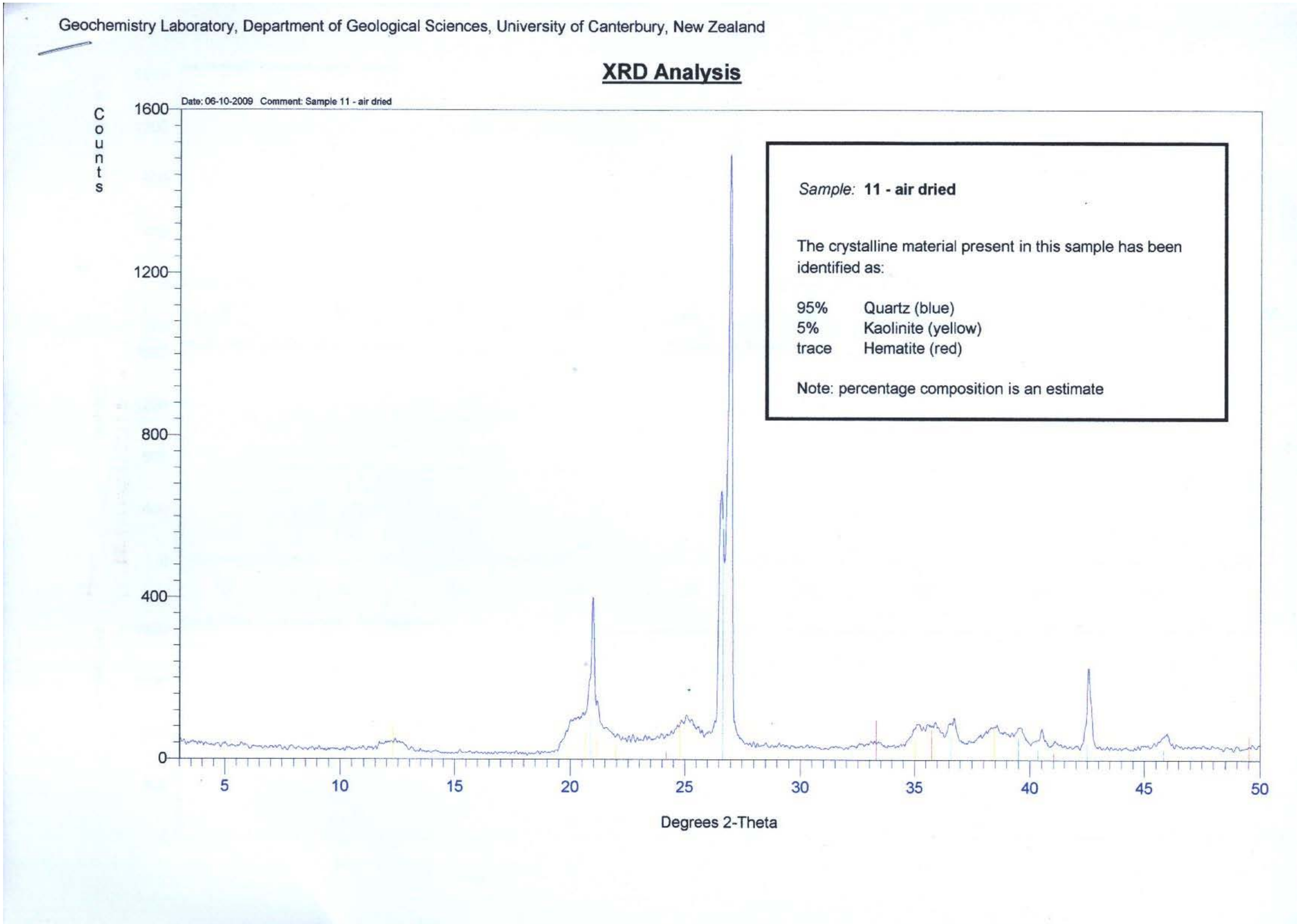


Sample: **10 - air dried**

The crystalline material present in this sample has been identified as:

95%	Quartz (blue)
5%	Kaolinite (yellow)
trace	Hematite (red)

Note: percentage composition is an estimate



Geochemistry Laboratory, Department of Geological Sciences, University of Canterbury, New Zealand
XRD Analysis
Date: 06-10-2009 Comment: Sample 11 - air dried
Counts
1600
1200
800
400
0
5
10
15
20
25
30
35
40
45
50
Degrees 2-Theta
Sample: 11 - air dried
The crystalline material present in this sample has been identified as:
95% Quartz (blue)
5% Kaolinite (yellow)
trace Hematite (red)
Note: percentage composition is an estimate

Namuka I Lau Falling Head Test

Falling Head Test 1

Minutes	hrs	Day	Real Time	Level (cm)	level (m)
0		28/2	1427	77	0.77
1		28/2	1428	77	0.77
10	0.167	28/2	1437	76.9	0.769
100	1.67	28/2	1607	76.6	0.766
1000	16.67	3-Jan	707	76.2	0.762

K=(2/3*(a/A)*(L/t)*log(ho/h))

a=		1.67	0.000133	0.000133	0.00013 3	0.00013 3
A=	hrs		0.000154	0.000154	0.00015 4	0.00015 4
L=		0.11	0.11	0.11	0.11	0.11
t=			1000	100	10	1
h0=			0.77	0.77	0.77	0.77
h=			0.762	0.766	0.769	0.77
	k=		4.78E-09	2.38E-08	5.95E-08	0
	average k =		2.78E-08			

Wainigasau Falling Head Test

Falling Head Test 1

Minutes	hrs	Day	Real Time	Level (cm)	level (m)
0		28/2	1427	80	0.8
1		28/2	1428	79.9	0.799
10	0.167	28/2	1437	79	0.79
100	1.67	28/2	1607	69.9	0.699
1000	16.67	3-Jan	707	20.02	0.2002
			2036	49.1	0.491

Falling Head Test 2

Minutes	hrs	Day	Real Time	Level (cm)	level (m)
0		3-Jan	710	87.8	0.878
1		3-Jan	711	87.6	0.876
10	0.167	3-Jan	720	86.6	0.866
100	1.67	3-Jan	850	78.2	0.782
1000	16.67	3-Jan	1150	32.2	0.322

K=(2/3*(a/A)*(L/t)*log(ho/h))

a=	132.665		0.000133	0.000133	0.00013 3	0.00013 3
A=	153.86		0.000154	0.000154	0.00015 4	0.00015 4
L=	0.11		0.11	0.11	0.11	0.11
t=			1000	100	10	1
h0=			0.8	0.8	0.8	0.8
h=			0.2002	0.699	0.79	0.799
	k=		6.34E-07	6.18E-07	5.76E-07	5.72E-07
	average k =		6E-07			
d (mm)		14	0.014			
r (mm)		7	0.007			
a (mm^2)		153.86	0.000154			
d (mm)		13	0.013			
r (mm)		6.5	0.0065			
a (mm^2)		132.665	0.000133			
a=		0.000133	0.000133	0.000133	0.00013 3	0.00013 3
A=		0.000154	0.000154	0.000154	0.00015 4	0.00015 4
L=	0.11		0.11	0.11	0.11	0.11
t=			1000	100	10	1
h0=			0.878	0.878	0.878	0.878
h=			0.322	0.782	0.866	0.876

k=	4.59E-07	5.3E-07	6.3E-07	1.04E-06
average k =	6.66E-07			

Appendix A.2

Determination of Bulk Unit Weight and Water Content

Water Content Determination:

Namuka i lau

$$w = \frac{m^2 - m^3}{m^3 - m^1} \times 100$$

$$w = \frac{0.099 - 0.0756}{0.0756 - 0.026} \times 100$$

$$\mathbf{w = \underline{47.18\%}}$$

Wainigasau

$$w = \frac{m^2 - m^3}{m^3 - m^1} \times 100$$

$$w = \frac{0.468 - 0.331}{0.331 - 0.026} \times 100$$

$$\mathbf{w = \underline{44.92\%}}$$

Bulk Unit Weight

Nmauka i lau

The bulk unit weight, γ_b, determined for Namuka i lau was calculated as follows:

$$\gamma_b = \frac{\text{total weight}}{\text{total volume}} = \frac{m_t g}{v_t}$$

$$\gamma_b = \frac{(0.073)}{(0.052)(0.038)(0.025)} \times \frac{9.81}{1000}$$

$$\gamma_b = \frac{0.073}{0.0000494} \times \frac{9.81}{1000}$$

$$\gamma_b = 1477.73 \times \frac{9.81}{1000}$$

$$\boldsymbol{\gamma_b} = \underline{\mathbf{14.5\ kN/m^3}}$$

Wainigasau

The bulk unit weight, γ_b, determined for Wainigasau was calculated as follows:

$$\gamma_b = \frac{\text{total weight}}{\text{total volume}} = \frac{m_t g}{v_t}$$

$$\gamma_b = \frac{(0.443)}{(0.073)(0.063)(0.06)} \times \frac{9.81}{1000}$$

$$\gamma_b = \frac{0.443}{0.000276} \times \frac{9.81}{1000}$$

$$\gamma_b = 1605.42 \times \frac{9.81}{1000}$$

$$\boldsymbol{\gamma_b} = \underline{\mathbf{15.8\ kN/m^3}}$$

Appendix A.3

Atterburg Limits Datasheets

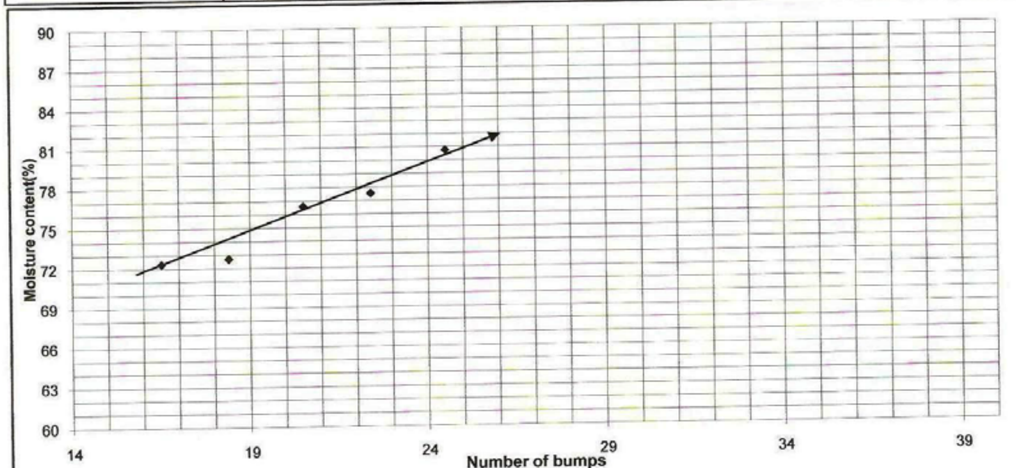
DEPARTMENT OF NATIONAL ROAD
ROADS LABORATORY
SAMABULA

Department of National Roads

Atterberg Limit

Company: Wainigasau (1) MRD 01210101		Job ref	
Location: Wainigasau (1) MRD 01210101		Borehole/Pit no.	
Soil Description: Red Clay		Sample no.	200906
		Depth:	…………………………
Test Method: P.I	BS1377: Part 2 1990:4 5/4 6*	Date:	16.03.2009
		Operator:	Pita/Veta

	Liquid Limit						Plastic Limit			
Test No.	1	2	3	4	5		1	2	3	Average
Container No.	53	38	85	43	56		22	70	3	
Number of bumps	16.50	18.40	20.5	22.40	24.5					
Weight of wet soil + container	24.74	26.70	28.89	30.68	32.12		24.47	24.47	24.47	
Weight of dry soil + container	17.86	19.40	20.25	20.98	21.46		18.69	18.41	18.73	
Weight of container	8.35	9.36	8.97	8.47	8.26		9.24	8.27	9.42	
Weight of water	6.88	7.30	8.64	9.70	10.66		5.78	6.06	5.74	
Weight of dry soil	9.51	10.04	11.28	12.51	13.20		9.45	10.14	9.31	
Moisture content	72.3	72.7	76.6	77.5	80.8		61.2	59.8	61.7	60.9



Sample preparation*
as reveived washed on 425µm sieve
air dried at ………..°C
oven dried at ……..°C not known

µm sieve………..%

Liquid Limit	75.9%
Plastic Limit	60.9%
Plastic Index	15.0%

*Delete as appropriate
Linear Shrinkage Limit 150 -125 = x 100 = 16.7
150

Tested by: Pita, Veta ………… Checked by: ……………

Approved by: ……………
Date: 20-03-09

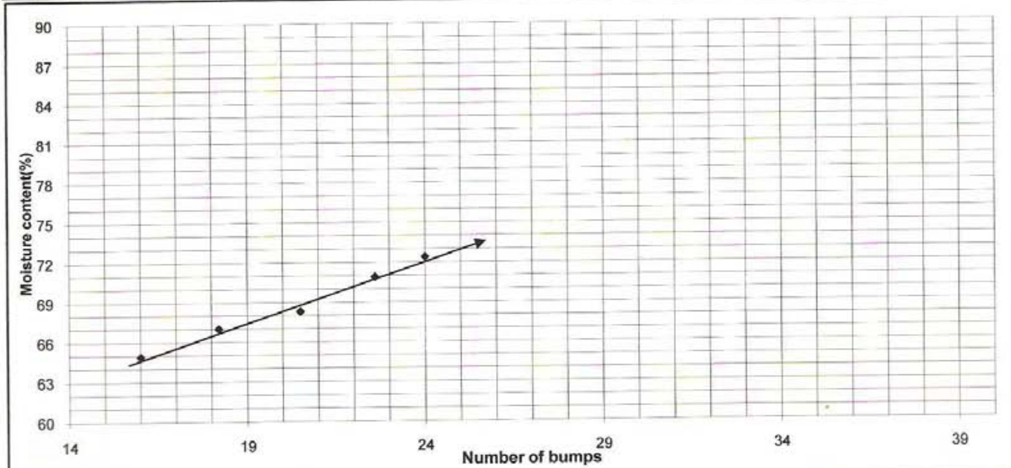
DEPARTMENT OF NATIONAL ROAD
ROADS LABORATORY
SAMABULA

Department of National Roads

Atterberg Limit

Company: MRD		Job ref	
Location: Namuka - I - Lau (2) MRD 02210102		Borehole/Pit no.	
Soil Description: Red Clay		Sample no.	200905
		Depth:	....
Test Method: P.I	BS1377: Part 2 1990:4 5/4 6*	Date:	13.03.2009
		Operator:	Kalesi/Susana/Sera/Lo

	Liquid Limit						Plastic Limit			
Test No.	1	2	3	4	5		1	2	3	Average
Container No.	19A	35	83	71	84		59	94	11	
Number of bumps	16.00	18.20	20.5	22.60	24.0					
Weight of wet soil + container	30.76	31.45	32.48	33.98	34.26		29.31	29.36	29.09	
Weight of dry soil + container	21.88	22.21	22.72	23.46	23.79		22.60	22.66	22.44	
Weight of container	8.20	8.42	8.43	8.62	9.33		9.36	9.42	9.46	
Weight of water	8.88	9.24	9.76	10.52	10.47		6.71	6.70	6.65	
Weight of dry soil	13.68	13.79	14.29	14.84	14.46		13.24	13.24	12.98	
Moisture content	64.9	67.0	68.3	70.9	72.4		50.7	50.6	51.2	50.8



Sample preparation*
as reveived washed on 425μm sieve
air dried at ....°C
oven dried at ....°C not known

μm sieve..........%

Liquid Limit	68.3%	*Delete as appropriate
Plastic Limit	50.8%	Linear Shrinkage Limit
Plastic Index	17.5%	

$\frac{150 - 137}{150} = \times 100 = 8.7$

Tested by: Kalesi .... Checked by: ....

Approved by: ....

Date: 16 - 03 - 09

Appendix A.4

Grainsize Analysis Datasheets

PUBLIC WORKS DEPARTMENT
SAMABULA LABORATORY
SIEVE ANALYSIS

Lab.No.	200905
Operator.	Abin/Mala/Krishneel
Date:.	13.03.2009

Road : Queens Road **Section:** MRD 02210102

Location: Namuka-i-Lau (2) **Material Type:** Red Clay

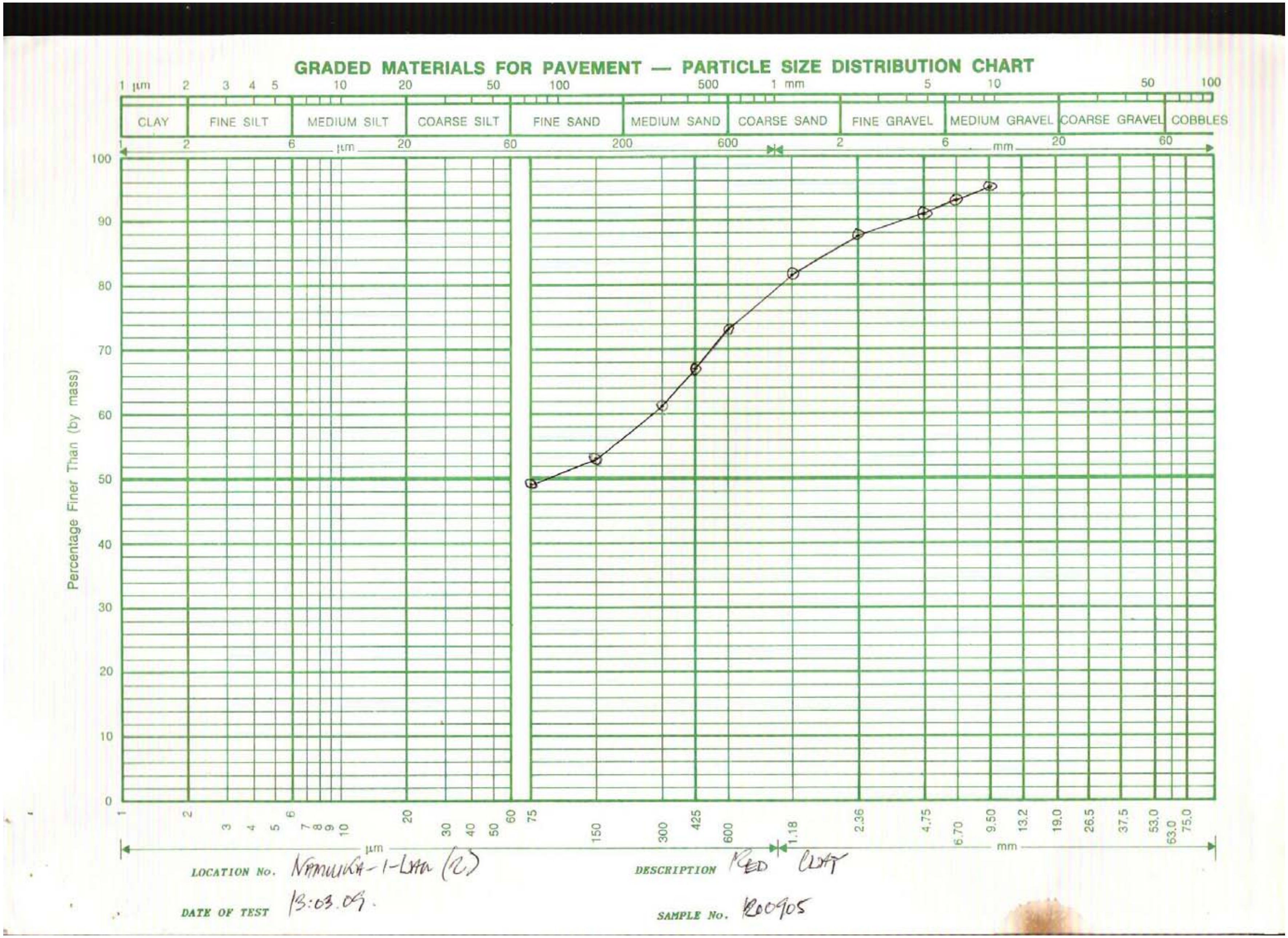
TOTAL SAMPLE			
Sieving procedure Wet/Dry	1303.7	g.M
Dry mass after washig	672.0	gM_1
Mass washing thru'75	631.70	gM
SPLIT SAMAPLE		
Mass passing last sieve		gM_3
Mass after spliting		gM_4

Spliting Factor = $-\frac{M_3}{M_4}$

Sieve Overload Mass g	Sieve Size	Mass Retained g	Total Mass Passing g	% of Total Passing	Mass Retained g	Corrected Mass Retained g	Total Mass Passing g	% of Total Passing
300mm Diam	**75mm**							
	53mm							
2200	**37.5mm**							
1800	**26.5mm**							
1200	**19.0mm**							
900	**13.2mm**			100%				
600	**9.50mm**	67.7	1236.0	94.8				
500	**6.70 mm**	23.9	1212.1	93.0				
200 mm 400 Diam.	**4.75 mm**	27.7	1184.4	90.8				
150	**2.36mm**	41.9	1142.5	87.6				
100	**1.18 mm**	76.6	1065.9	81.8				
75	**600 um**	117.9	948	72.7				
60	**425 um**	74.8	873.2	67.0				
50	**300 um**	72.3	800.9	61.4				
40	**150 um**	106.8	694.1	53.2				
25	**75 um**	55.5	638.6	49.0				
	Pan Total	6.9 672.0	631.7	48.5				

Tested by: Abin, Mala Checked by: [signature] Approved by: [signature]

Date: 16-03-09



GRADED MATERIALS FOR PAVEMENT — PARTICLE SIZE DISTRIBUTION CHART
CLAY
FINE SILT
MEDIUM SILT
COARSE SILT
FINE SAND
MEDIUM SAND
COARSE SAND
FINE GRAVEL
MEDIUM GRAVEL
COARSE GRAVEL
COBBLES
Percentage Finer Than (by mass)
µm
mm
LOCATION No. NAMUKA-I-LAU (2)
DATE OF TEST 13:03.09.
DESCRIPTION RED CLAY
SAMPLE No. R00905

PUBLIC WORKS DEPARTMENT
SAMABULA LABORATORY
SIEVE ANALYSIS

Lab.No.	200906
Operator.	Abin/Mala/Krishneel
Date:.	13.03.2009

Road :

Section: MRD 02210101

Location: Wainigasau (1)

Material Type: Red Clay

Sieve Overload Mass g	Sieve Size	Mass Retained g	Total Mass Passing g	% of Total Passing
300mm Diam	**75mm**			
	53mm			
2200	**37.5mm**			
1800	**26.5mm**			
1200	**19.0mm**			
900	**13.2mm**			
600	**9.50mm**			
500	**6.70 mm**			
200 mm 400 Diam.	**4.75 mm**			100%
150	**2.36mm**	53.6	1269.2	95.9
100	**1.18 mm**	151.0	1118.2	84.5
75	**600 um**	229.2	889	67.2
60	**425 um**	89.2	799.8	60.5
50	**300 um**	60.5	739.3	55.9
40	**150 um**	96.8	642.5	48.6
25	**75 um**	48.9	593.6	44.9
	Pan Total	15.8 745.0	577.8	43.7

TOTAL SAMPLE

Sieving procedure Wet/Dry	1322.8	g.M
Dry mass after washig	745.1	gM_1
Mass washing thru'75	577.70	gM

SPLIT SAMAPLE

Mass passing last sieve		gM_3
Mass after spliting		gM_4

Spliting Factor = $-\frac{M_3}{M_4}$

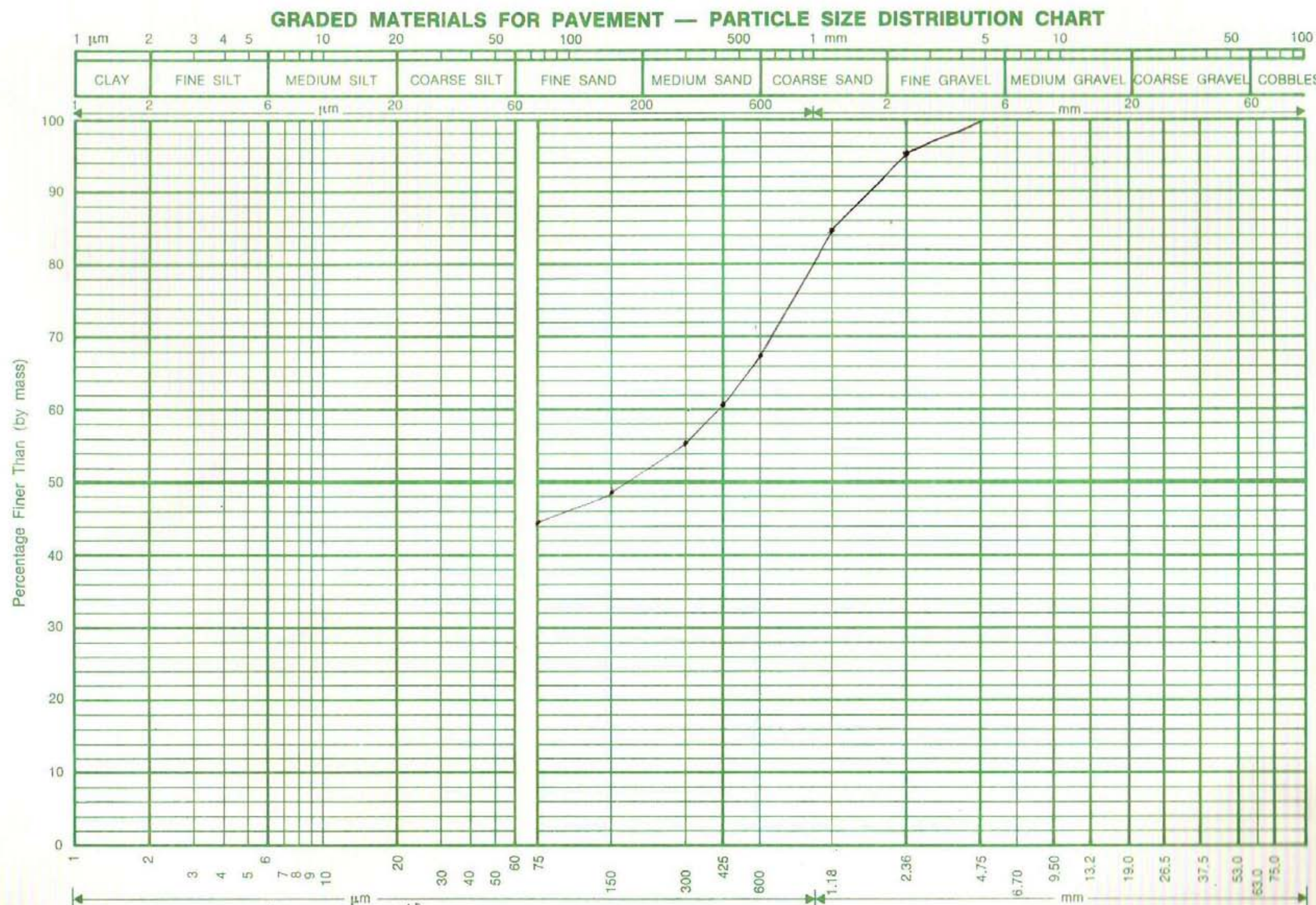
Mass Retained g	Corrected Mass Retained g	Total Mass Passing g	% of Total Passing

Tested by: Abin, Mala, Krishneel

Checked by:

Approved by:

Date: 16-03-09



LOCATION No. WANIGASHA (1)

DESCRIPTION RED CLAY

DATE OF TEST 13:03:09

SAMPLE No. 20090C

Appendix A.5

Falling Head Test Calculations

Namuka I Lau Falling Head Test

	Falling Head Test 1				
Minutes	**hrs**	**Day**	**Real Time**	**Level (cm)**	**level (m)**
0		28/2	1427	77	0.77
1		28/2	1428	77	0.77
10	0.167	28/2	1437	76.9	0.769
100	1.67	28/2	1607	76.6	0.766
1000	16.67	3-Jan	707	76.2	0.762

K=(2/3*(a/A)*(L/t)*log(ho/h))

a=		1.67	0.000133	0.000133	0.000133	0.000133
A=	hrs		0.000154	0.000154	0.000154	0.000154
L=		0.11	0.11	0.11	0.11	0.11
t=			1000	100	10	1
h0=			0.77	0.77	0.77	0.77
h=			0.762	0.766	0.769	0.77
	k=		4.78E-09	2.38E-08	5.95E-08	0
	average k =		2.78E-08			

Wainigasau Falling Head Test

	Falling Head Test 1				
Minutes	**hrs**	**Day**	**Real Time**	**Level (cm)**	**level (m)**
0		28/2	1427	80	0.8
1		28/2	1428	79.9	0.799
10	0.167	28/2	1437	79	0.79
100	1.67	28/2	1607	69.9	0.699
1000	16.67	3-Jan	707	20.02	0.2002
			2036	49.1	0.491
	Falling Head Test 2				
Minutes	**hrs**	**Day**	**Real Time**	**Level (cm)**	**level (m)**
0		3-Jan	710	87.8	0.878
1		3-Jan	711	87.6	0.876
10	0.167	3-Jan	720	86.6	0.866
100	1.67	3-Jan	850	78.2	0.782
1000	16.67	3-Jan	1150	32.2	0.322

K=(2/3*(a/A)*(L/t)*log(ho/h))

a=	132.665	0.000133	0.000133	0.000133	0.000133
A=	153.86	0.000154	0.000154	0.000154	0.000154
L=	0.11	0.11	0.11	0.11	0.11
t=		1000	100	10	1
h0=		0.8	0.8	0.8	0.8
h=		0.2002	0.699	0.79	0.799

	k=	6.34E-07	6.18E-07	5.76E-07	5.72E-07
	average k =	6E-07			
d (mm)	14	0.014			
r (mm)	7	0.007			
a (mm^2)	153.86	0.000154			
d (mm)	13	0.013			
r (mm)	6.5	0.0065			
a (mm^2)	132.665	0.000133			
a=	0.000133	0.000133	0.000133	0.000133	0.000133
A=	0.000154	0.000154	0.000154	0.000154	0.000154
L=	0.11	0.11	0.11	0.11	0.11
t=		1000	100	10	1
h0=		0.878	0.878	0.878	0.878
h=		0.322	0.782	0.866	0.876
	k=	4.59E-07	5.3E-07	6.3E-07	1.04E-06
	average k =	6.66E-07			

Appendix A.6

Definitions: Factor of Safety and Methods of Analysis

1.1 FACTOR OF SAFETY

A primary objective of slope stability analysis is estimation of a factor of safety for the considered slope and slide mass. An intuitive formulation of a factor of safety appropriate to translational sliding is the ration of resisting forces to driving forces acting parallel to the direction of translation. Thus for translational slides:

$$FS = \frac{R}{D}$$

where R and D are the total resisting and driving forces. The sign convention for driving and resisting forces are simple in that forces that act downhill in the direction of the potential slide are driving forces and the uphill forces are resisting. A factor of safety less than one indicates a positive, downhill acceleration. A factor of safety greater than one indicates a negative acceleration that for a slide mass at rest is physically meaningless so the slide mass would be stable and remain at rest. This defined factor of safety is a global factor of safety in contrast to a local factor of safety such as that defined below, which is a ration of strength to stress at a point. A local factor of safety less than one indicates the elastic limit may be exceeded; yielding and failure ensue, at the considered point. A global factor of safety less than one implies yielding and failure over an extended failure surface and is indicative of collapse (Pariseau, 2006).

In the analysis of slope stability, the factor of safety can be generally defined as :

$$FS = \frac{\tau f}{\tau d}$$

where Fs = factor of safety with respect to strength

τf = average shear strength of the soil

τd = average shear stress developed along the potential failure surface

The shear strength of a soil consists two components of cohesion and friction and is written as:

$$\tau f = c' + \sigma' tan\emptyset'$$

where c' = cohesion

Ø' = angle of friction

σ' = normal stress on the potential failure surface

Similarly

$$\tau d = c'd + \sigma' tan\emptyset' d$$

Our factor of safety can now be written as:

$$Fs = \frac{c' + \sigma' tan\emptyset'}{c'd + tan\emptyset' d}$$

The factor of safety with respect to friction and cohesion can be defined as:

$$Fc' = \frac{c'}{c'd}$$

and

$$F\emptyset' = \frac{tan\emptyset'}{tan\emptyset' d}$$

When Fc' becomes equal to FØ', it gives the factor of safety with respect to strength and we can write

$$Fs = Fc' = F\emptyset'$$

A factor of safety equal to 1 represents a slope in a state of impending failure. A value equal or greater than 1.5 is acceptable for the design of a stable slope and acceptable for development (Das, 2002). For typical slope designs, the required factors of safety (nonseismic) are usually in the 1.25 to 1.5 range. Higher factors of safety may be required if there is a high risk of loss of life, infrastructure or high uncertainty regarding the pertinent design parameters. Similarly a lower factor of safety may be used if the engineer is confident of the accuracy of the input data and construction is monitored closely (Abramson et al, 2002).

1.2 METHODS OF ANALYSIS

Stability analysis of slopes by mathematical procedures is applicable only to the evaluation of failure by sliding along some definable surface. Currently this excludes the mathematical assessment of avalanches, flows, ash falls and progressive failures.

Slide failure occurs when the shearing resistance available along some failure surface in a slope is exceeded by shearing stresses available along some failure surface. Static analysis of sliding requires intricate knowledge of the location and shape of the potential failure surface, the shear strength along the failure surface, and the magnitude of the driving forces. Statically determinate failure forms may be classified as:

- infinite slope – translation on a plane parallel to the ground surface whose length is large compared with it depth below the surface (end effects can be neglected) (Morgenstern and Sangrey, 1978)

- Finite slope, planar surface – displacement of one or more blocks, or wedge shaped bodies, along planar surfaces with finite lengths

- Finite slope, curved surface - rotation along a curved surface approximated by a circular arc, log-spiral, or other definable cylindrical shape (Hunt, 2005)

In a finite slope, when a failure occurs in such a way that the failure surface intersects at or above the toe of the slope; it is termed a slope failure. The failure circle is termed a toe circle if it passes through the toe of the slope and a slope circle if it passes above the toe. A base failure occurs when the failure surface occurs at some distance below the toe of the slope and the failure circle is termed the midpoint circle.

1.2.1 Method of slices

The method of slices procedure for analyzing stability in a slope is firstly by dividing the soil above the failure surface into any number of parallel vertical slices and the stability of each slice is calculated separately. This technique takes into consideration the nonhomogeneity of the soils and associated pore pressures in soils and also takes into account the variation of the normal stress along the potential failure surface (Das, 2002).

Stability analysis in terms of effective stress can only be carried out when there is a reasonable assessment of pore water pressures within the slope. This is especially so for analyzing natural slopes and cuttings for the critical long term condition (Barnes, 1995).

The method of slices assumes a potential failure surface to be a circular arc with a radius R, and centre, O (Figure 5.1). The soil mass above the potential failure surface is divided into any number of vertical slices of width, b, and vertical height, h. For each of the slices, the base is assumed to be a straight line inclined at an angle, α, to the horizontal and with a length,

$$l = bsec \propto$$

Segmentation into slices is done only for convenience of analysis and the method is based on assumptions that the segment of soil (sum of slices)rotates around the centre of the circle, O, so that the factor of safety is the same for all the slices implying that there must be mutual support between slices, known as inter-slice forces and these are usually taken as normal and tangential to the sides of the slices.

The forces acting on a slice are:

1. The total weight of the slice, W, is calculated by the formula:

$$W = \gamma bh$$

where γ is the bulk unit weight of the soil

2. The weight of each slice will induce a shear force parallel to its base, $S = Wsin \propto$

3. The total normal force on the base, $N = \sigma l$

4. The total normal force is obtained from the total normal stress which two components, the effective normal stress, $N' = \sigma' l$, and the water force, $U = ul$, where, u, is the pore water pressure at the cents of the base of the slice.

5. The shearing resistance of the soil will provide a shear force, $T = \tau ml$

The interslice forces can be represented as total normal forces E_1 and E_2 and tangential shear forces X_1 and X_2. All external forces need to be considered in this analysis such as foundation, surcharge at toe or crest of the slope or a water filled tension crack.

Considering the moments about O, the centre of the potential failure surface, the sum of moments of the shear forces, T, on the failure must equal the moment of the weight of the segment of the soil mass. For any slice the lever arm of W, is $rsin\alpha$ so therefore:

$$\sum T_r = \sum Wrsin\alpha$$

Now,

$$T = \tau_m l = \frac{\tau_f}{F} l$$

therefore

$$\sum \frac{\tau_f}{F} l = \sum Wsin\alpha$$

therefore

$$F = \frac{\sum T_f\, l}{\sum Wsin\alpha}$$

In terms of effective stress, this can be written as,

$$F = \frac{\sum(c' + \sigma' tan\emptyset')l}{\sum Wsin\alpha}$$

or

$$F = \frac{c' L_a + tan\emptyset' \sum N'}{\sum Wsin\alpha}$$

where La is the arc length between A and B.

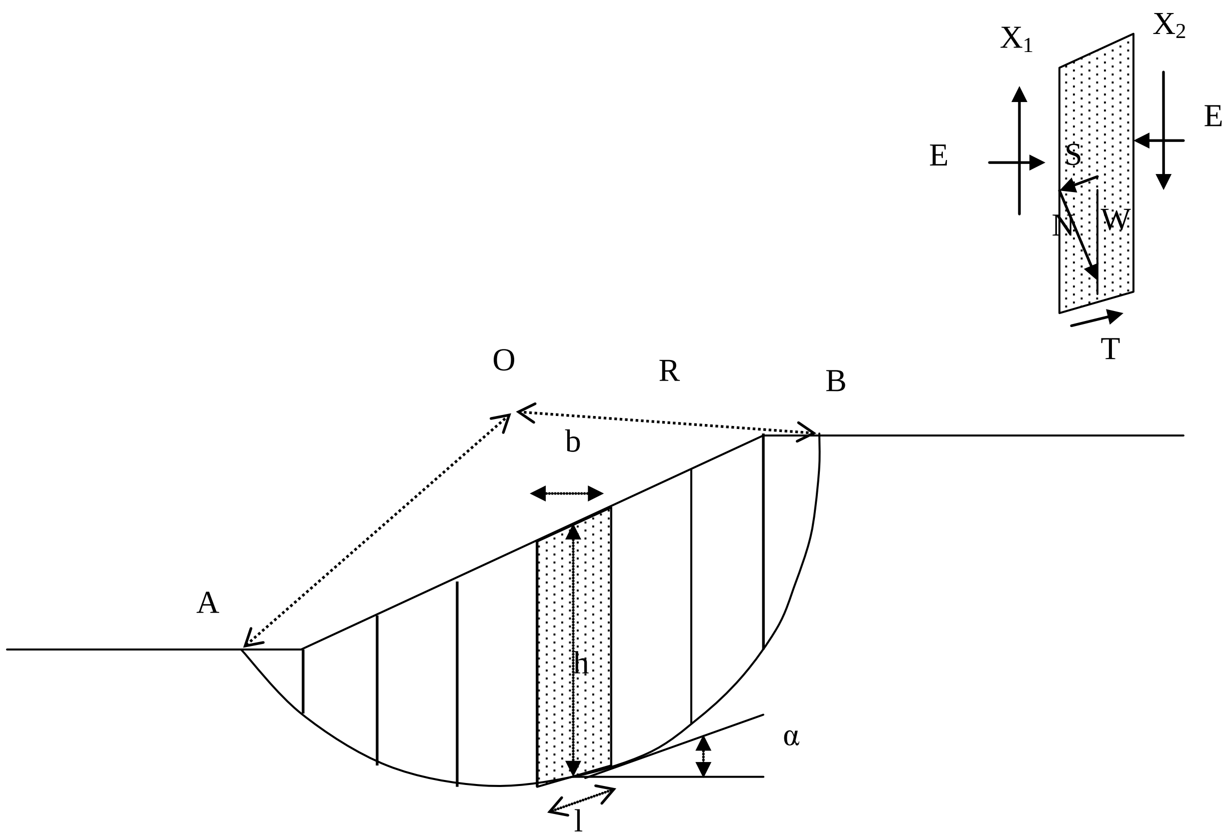


Figure 5.1 Dimensions and forces acting on a slope using the Method of Slices (adapted from Barnes, 1995)

6. The solution of the above equation requires assumptions to be made about the interslice forces which affects the values of the forces, 'N'.

The method used during our modeling process is the one defined below called the Bishop Simplified Method developed by A.W. Bishop in 1955.

5.4.2 Bishop Simplified Method

The Bishop Simplified method assumes that the tangential interslice forces are equal and opposite i.e.

$$X_1 = X_2$$

but the normal interslice forces are not equal i.e.

$$E_1 = E_2$$

Resolving forces in the vertical direction (ignoring E_1 and E_2) gives

$$W = N' cos \propto + ulcos \propto + \frac{c'l}{F} sin \propto + \frac{N'}{F} \tan\emptyset \sin\alpha$$

therefore

$$N' = \frac{W - \frac{c'l}{F} sin \propto - ulcos \propto}{cos \propto + \frac{tan\emptyset sin \propto}{F}}$$

If we insert this expression back into the equation for calculating the Factor of Safety using the method of slices we get,

$$F = \frac{1}{\sum Wsin \propto} \sum \frac{[c'b + (W - ub)tan\emptyset] sec \propto}{1 + \frac{tan \propto tan\emptyset}{F}}$$

The value of, F, occurs on both sides of the equation so a trial value of F must be chosen on the right hand side of the equation to obtain a value on the left hand side. By successive iteration convergence on the true value of, F, is obtained.

The use of computers is recommended when using this method as the factor of safety of a trial circle can be calculated in a matter of seconds and this speed of calculation is also beneficial in searching for the circle giving the lowest factor of safety, given that the software used will analyse circles over a grid of circle centres, with the circles passing through chosen points on the slope, over a range of radius values or tangential to particular levels. Software's also tend to allow for various complexities in soil profiles, groundwater conditions, external loadings and seismic effects (Barnes, 1995).

5.4.3 Janbu Method

The Janbu method is used to analyse non-circular failure surfaces such as those that may exist in non-homogeneous soil profiles such as with layered strata. The Janbu method is based on the assumption that the interslice forces are horizontal and there are no shear stresses between slices. This assumption alone almost always produces factors of safety that are smaller than those produced by more rigorous methods that satisfy complete equilibrium. The average factor of safety calculated using the Janbu method can be obtained using the following expression:

$$F = \frac{\sum[c'b + (W + dX - ub)tan\emptyset']\, m_{\propto}}{\sum(w + dX)tan \propto}$$

where

$$m_{\propto} = \frac{sec^2 \propto}{1 + \frac{tan\emptyset tan \propto}{F}}$$

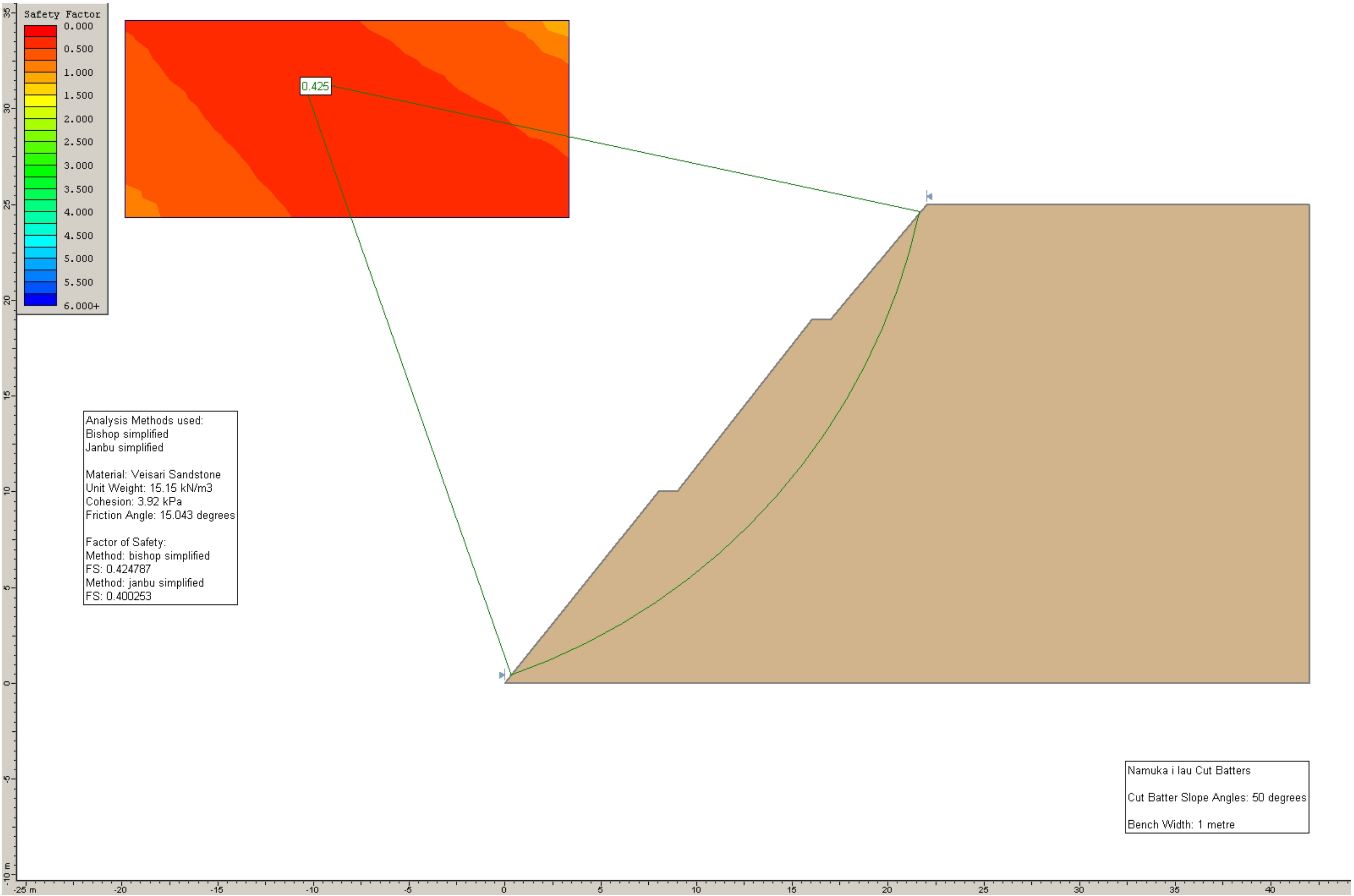
and $dX=X_1-X_2$, the resultant vertical interslice force.

Janbu also suggested correction factors to adjust the factors of safety to reasonable values. These correction factors are related to the depth of the slip mass and the soil type. This simplified Janbu method suggests the factor of safety F_o is obtained using the above expression and assuming the interslice forces can be ignored, dX = 0. The factor of safety, F including the influence of the interslice forces is then given by:

$$F = f_o F_o$$

Appendix A.7

Sensitivity Analysis: SLIDE computations for factor of safety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5.500
6.000+
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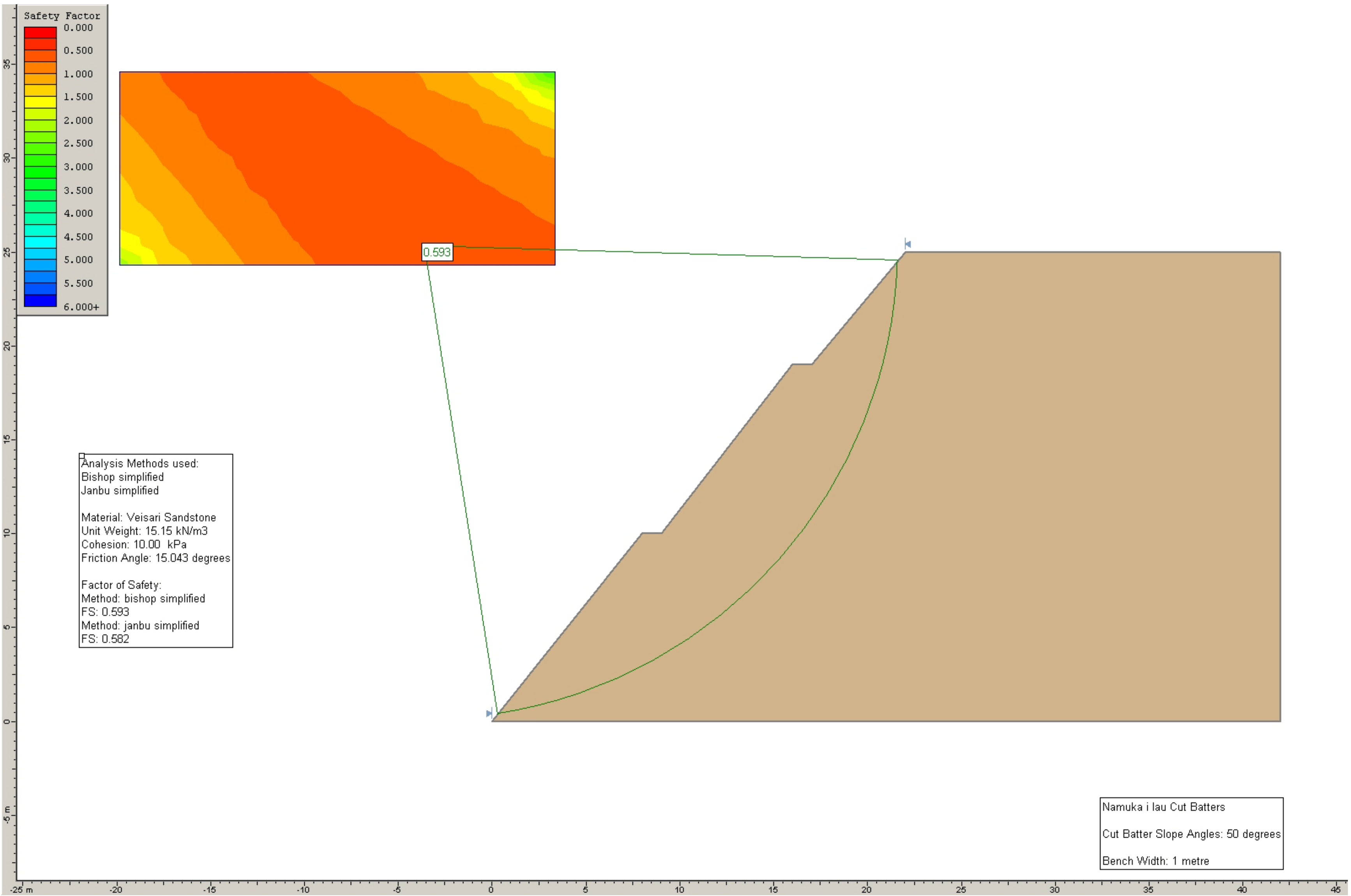
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 3.92 kPa
Friction Angle: 15.043 degrees
Factor of Safety:
Method: bishop simplified
FS: 0.424787
Method: janbu simplified
FS: 0.400253
Namuka i lau Cut Batters
Cut Batter Slope Angles: 50 degrees
Bench Width: 1 metre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6.000+
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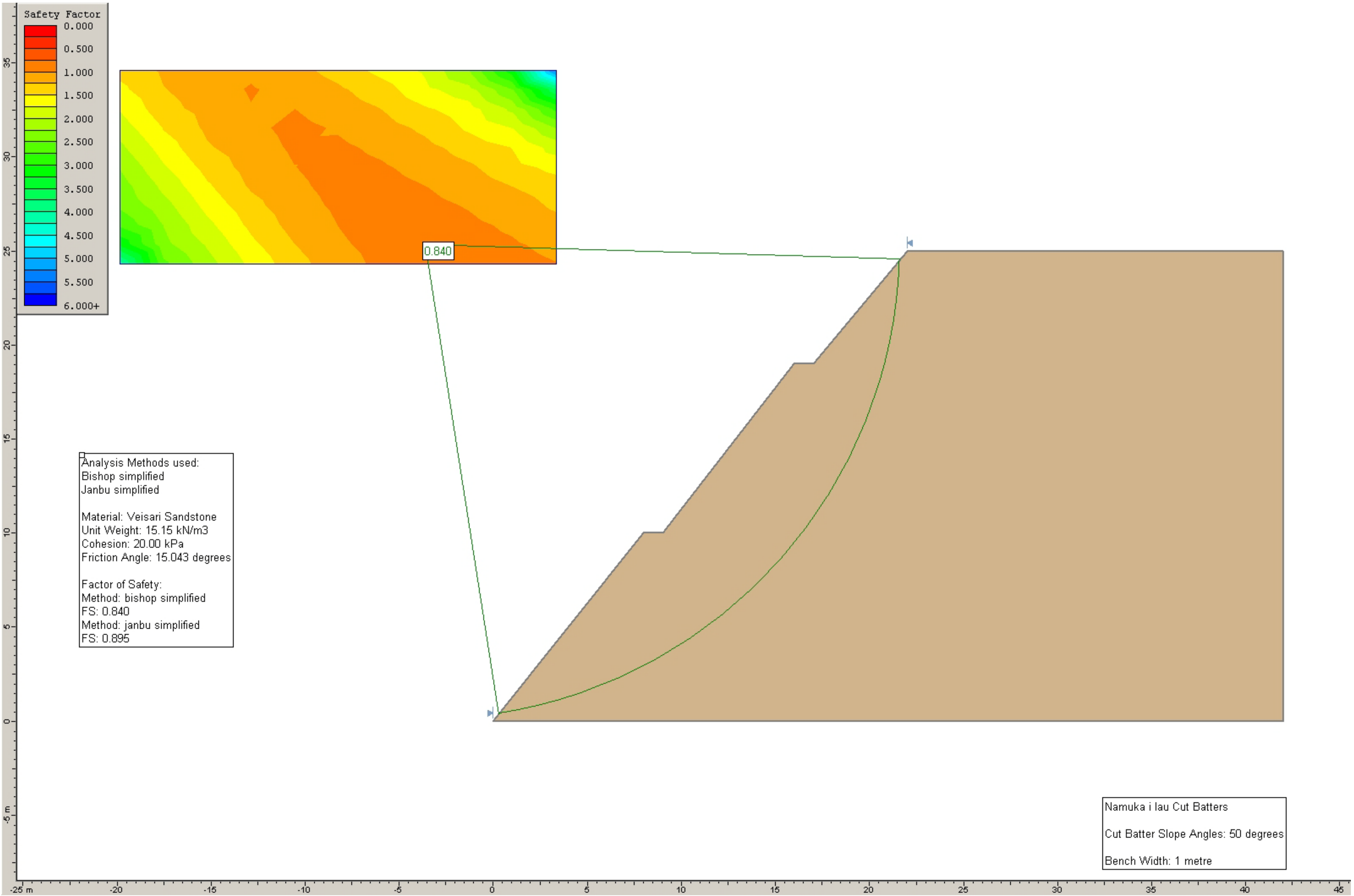
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 10.00 kPa
Friction Angle: 15.043 degrees
Factor of Safety:
Method: bishop simplified
FS: 0.593
Method: janbu simplified
FS: 0.582
Namuka i lau Cut Batters
Cut Batter Slope Angles: 50 degrees
Bench Width: 1 metre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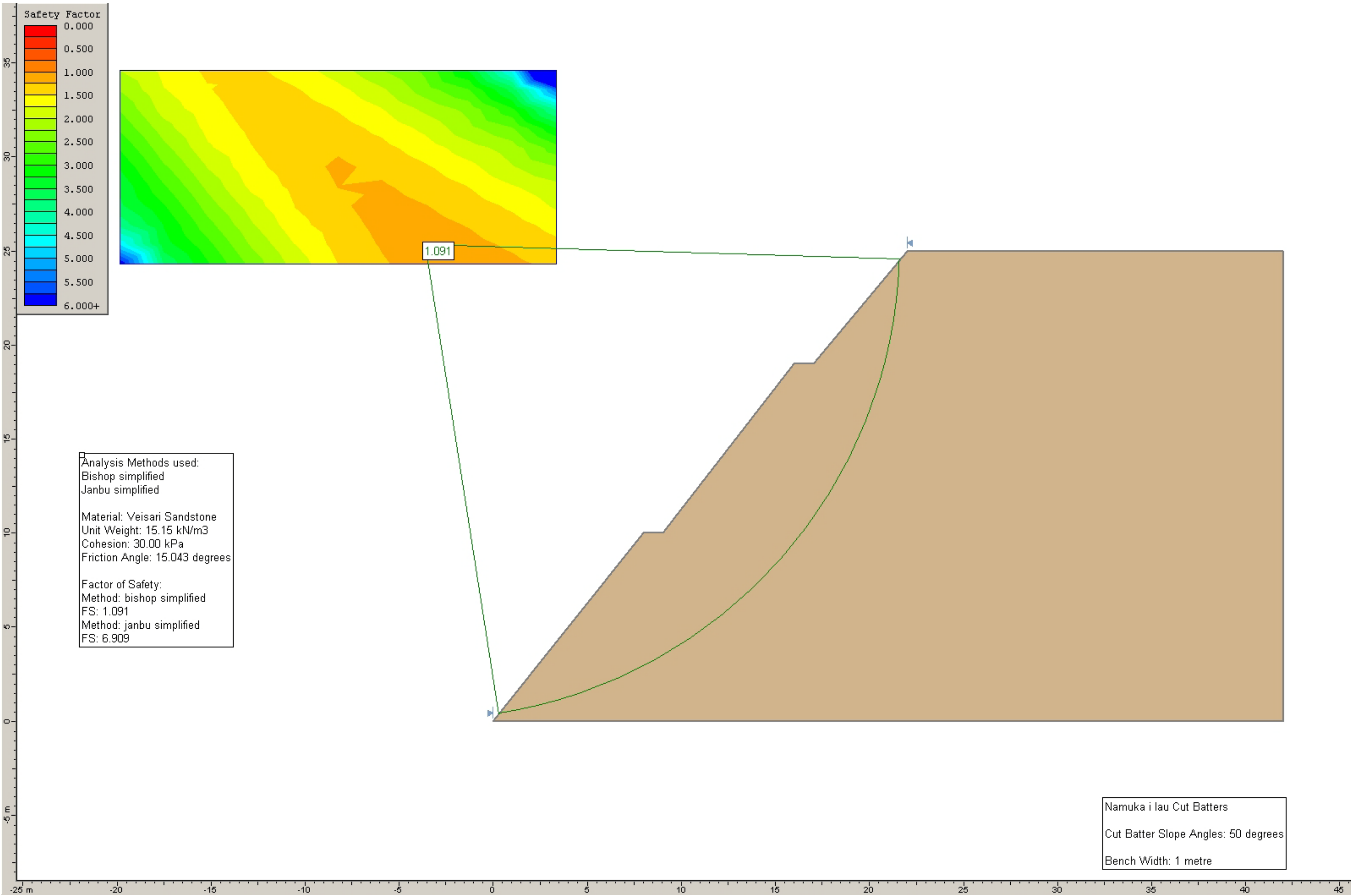
0.840
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 20.00 kPa
Friction Angle: 15.043 degrees
Factor of Safety:
Method: bishop simplified
FS: 0.840
Method: janbu simplified
FS: 0.895
Namuka i lau Cut Batters
Cut Batter Slope Angles: 50 degrees
Bench Width: 1 metre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5.500
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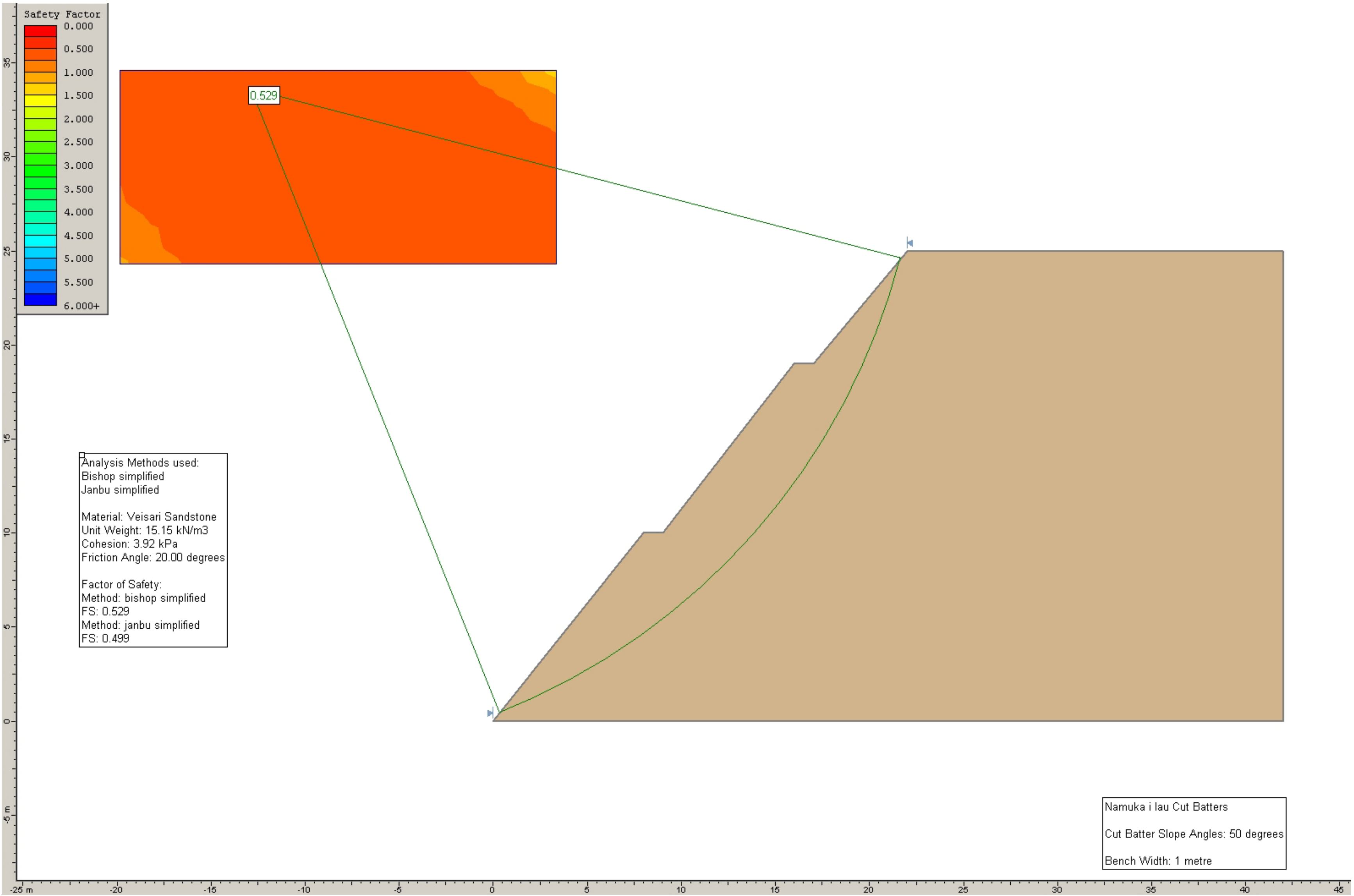
1.091
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 30.00 kPa
Friction Angle: 15.043 degrees
Factor of Safety:
Method: bishop simplified
FS: 1.091
Method: janbu simplified
FS: 6.909
Namuka i lau Cut Batters
Cut Batter Slope Angles: 50 degrees
Bench Width: 1 metre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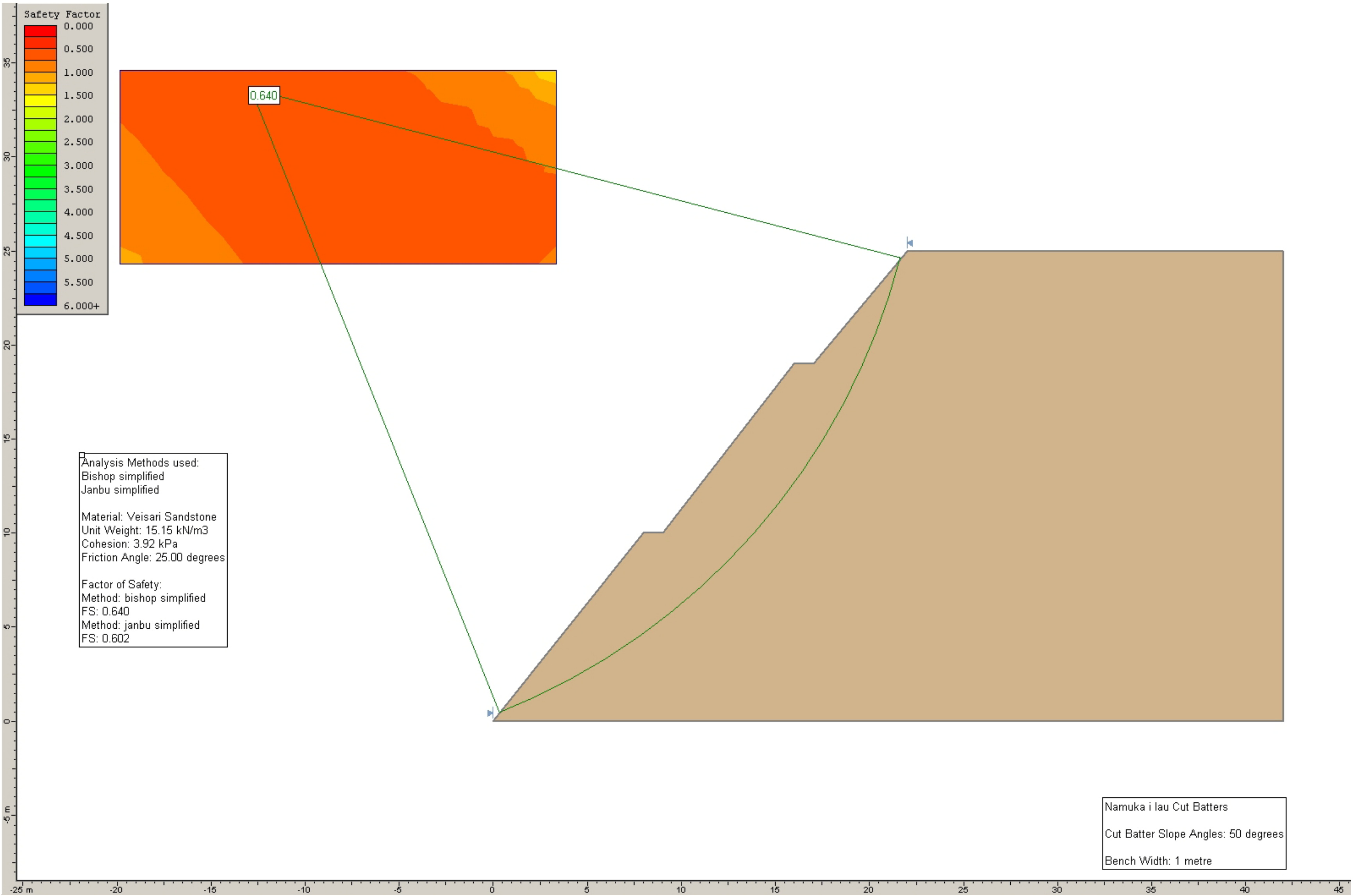
5.500
6.000+
0.529
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 3.92 kPa
Friction Angle: 20.00 degrees
Factor of Safety:
Method: bishop simplified
FS: 0.529
Method: janbu simplified
FS: 0.499
Namuka i lau Cut Batters
Cut Batter Slope Angles: 50 degrees
Bench Width: 1 metre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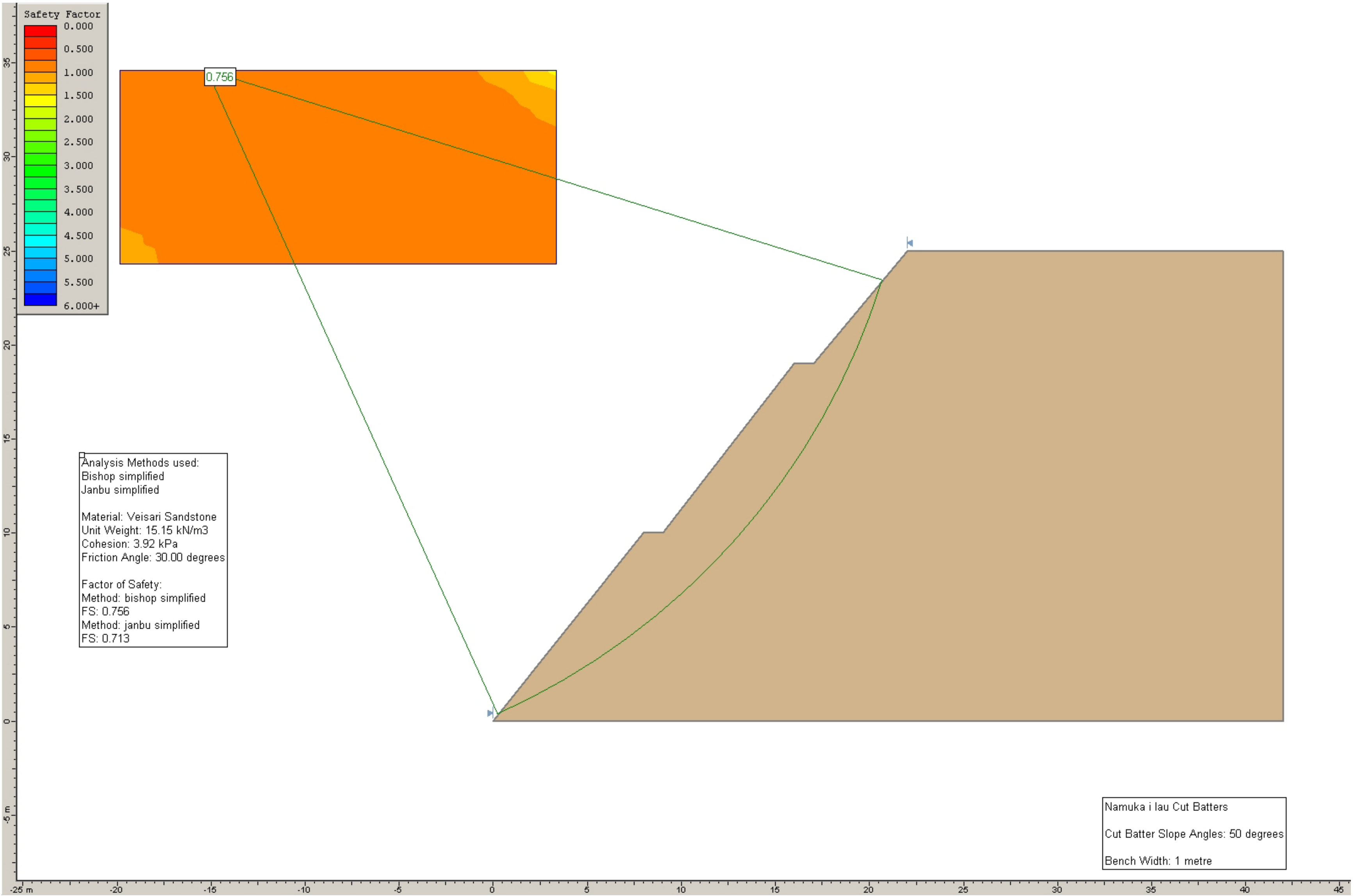
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 3.92 kPa
Friction Angle: 25.00 degrees
Factor of Safety:
Method: bishop simplified
FS: 0.640
Method: janbu simplified
FS: 0.602
Namuka i lau Cut Batters
Cut Batter Slope Angles: 50 degrees
Bench Width: 1 metre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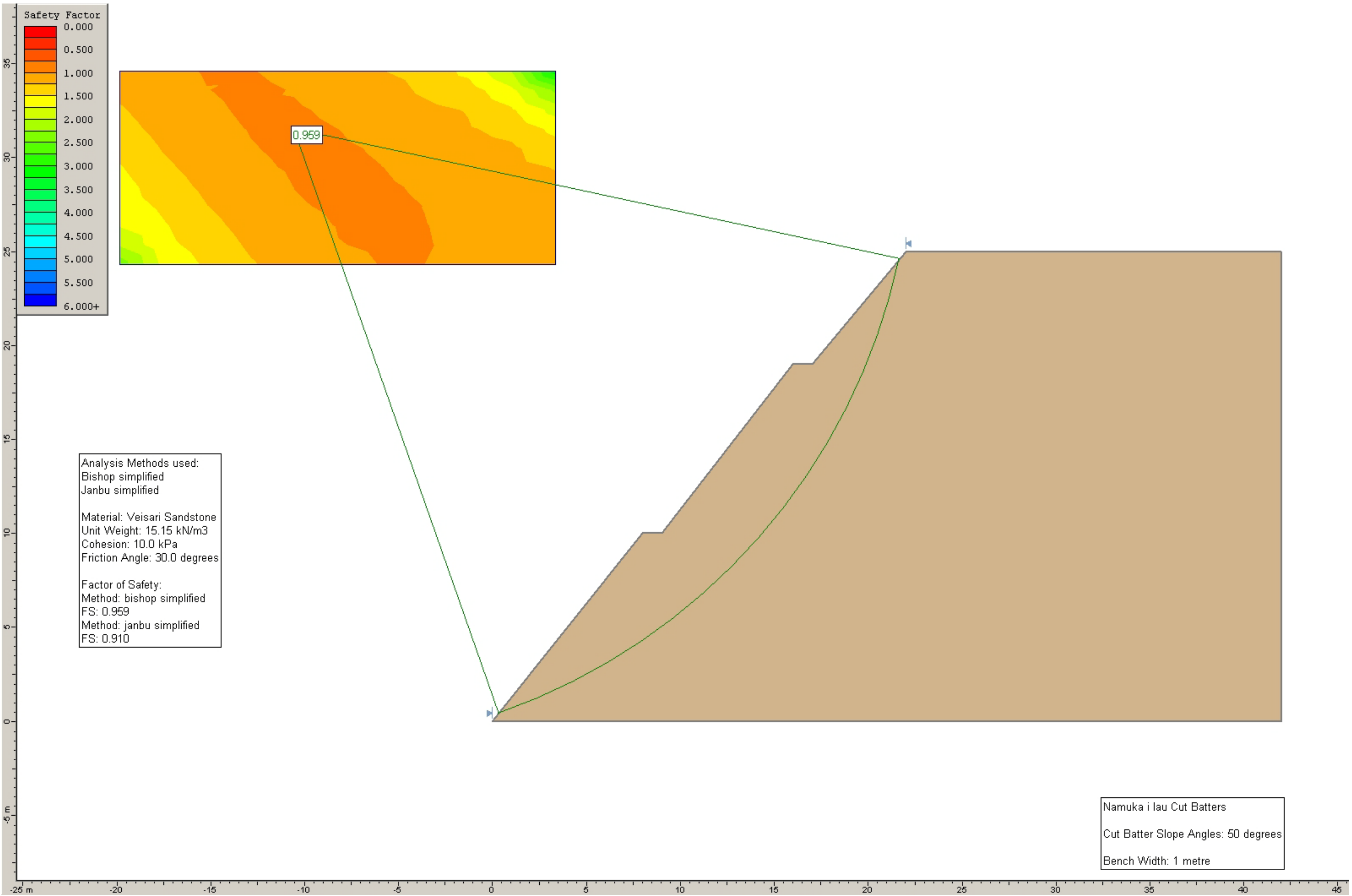
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 3.92 kPa
Friction Angle: 30.00 degrees
Factor of Safety:
Method: bishop simplified
FS: 0.756
Method: janbu simplified
FS: 0.713
Namuka i lau Cut Batters
Cut Batter Slope Angles: 50 degrees
Bench Width: 1 metre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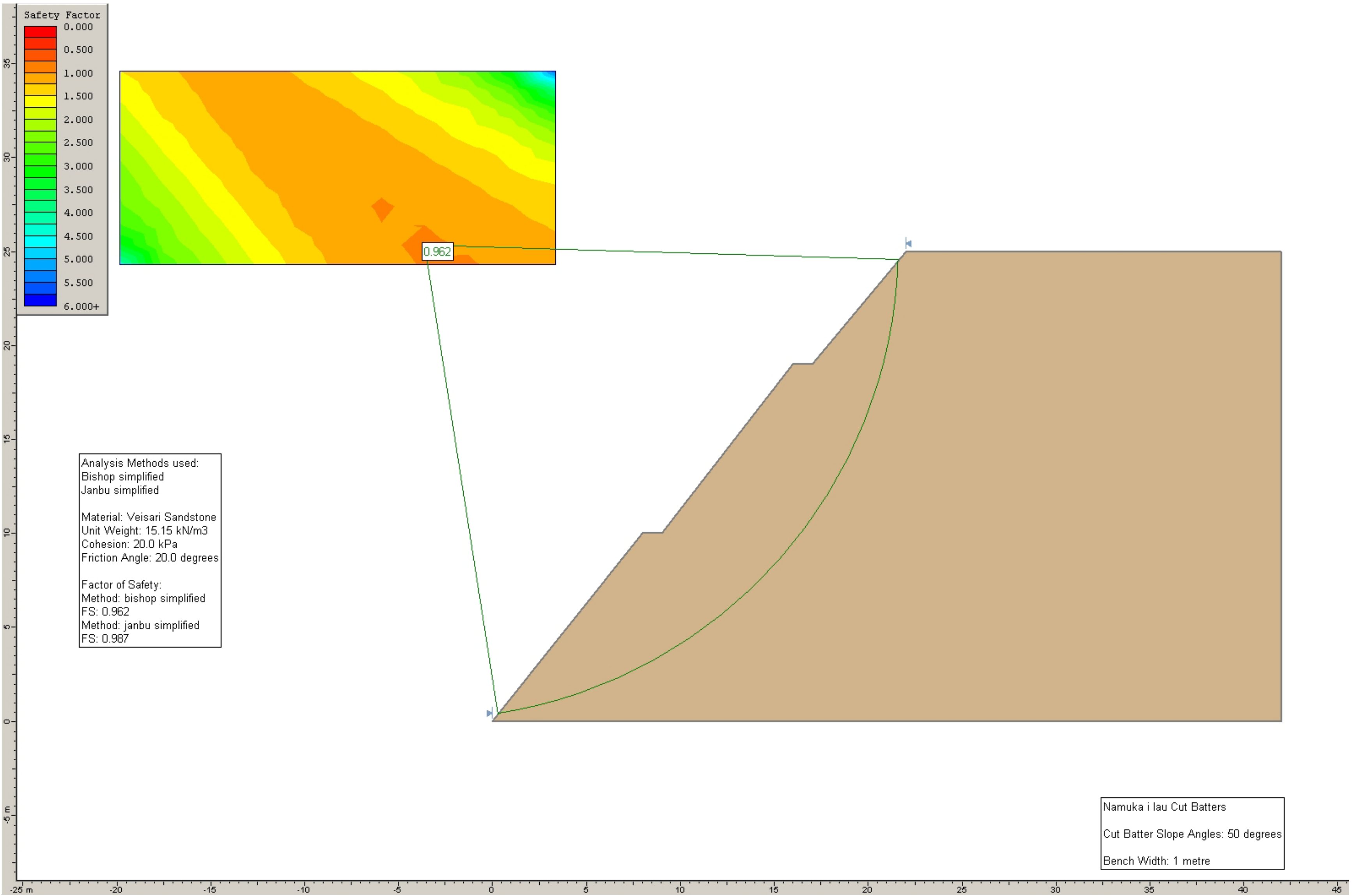
6.000+
0.959
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 10.0 kPa
Friction Angle: 30.0 degrees
Factor of Safety:
Method: bishop simplified
FS: 0.959
Method: janbu simplified
FS: 0.910
Namuka i lau Cut Batters
Cut Batter Slope Angles: 50 degrees
Bench Width: 1 metre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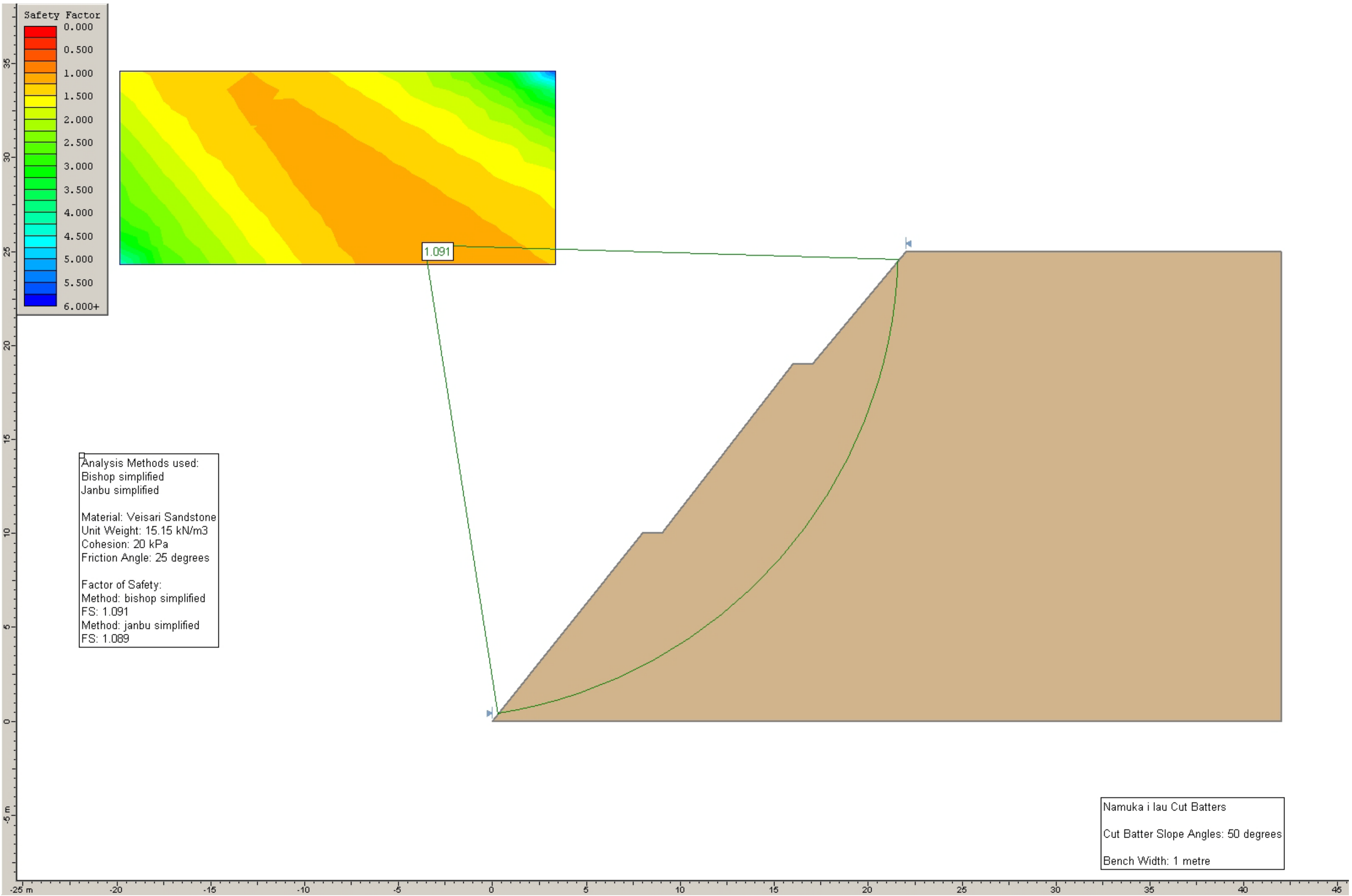
0.962
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 20.0 kPa
Friction Angle: 20.0 degrees
Factor of Safety:
Method: bishop simplified
FS: 0.962
Method: janbu simplified
FS: 0.987
Namuka i lau Cut Batters
Cut Batter Slope Angles: 50 degrees
Bench Width: 1 metre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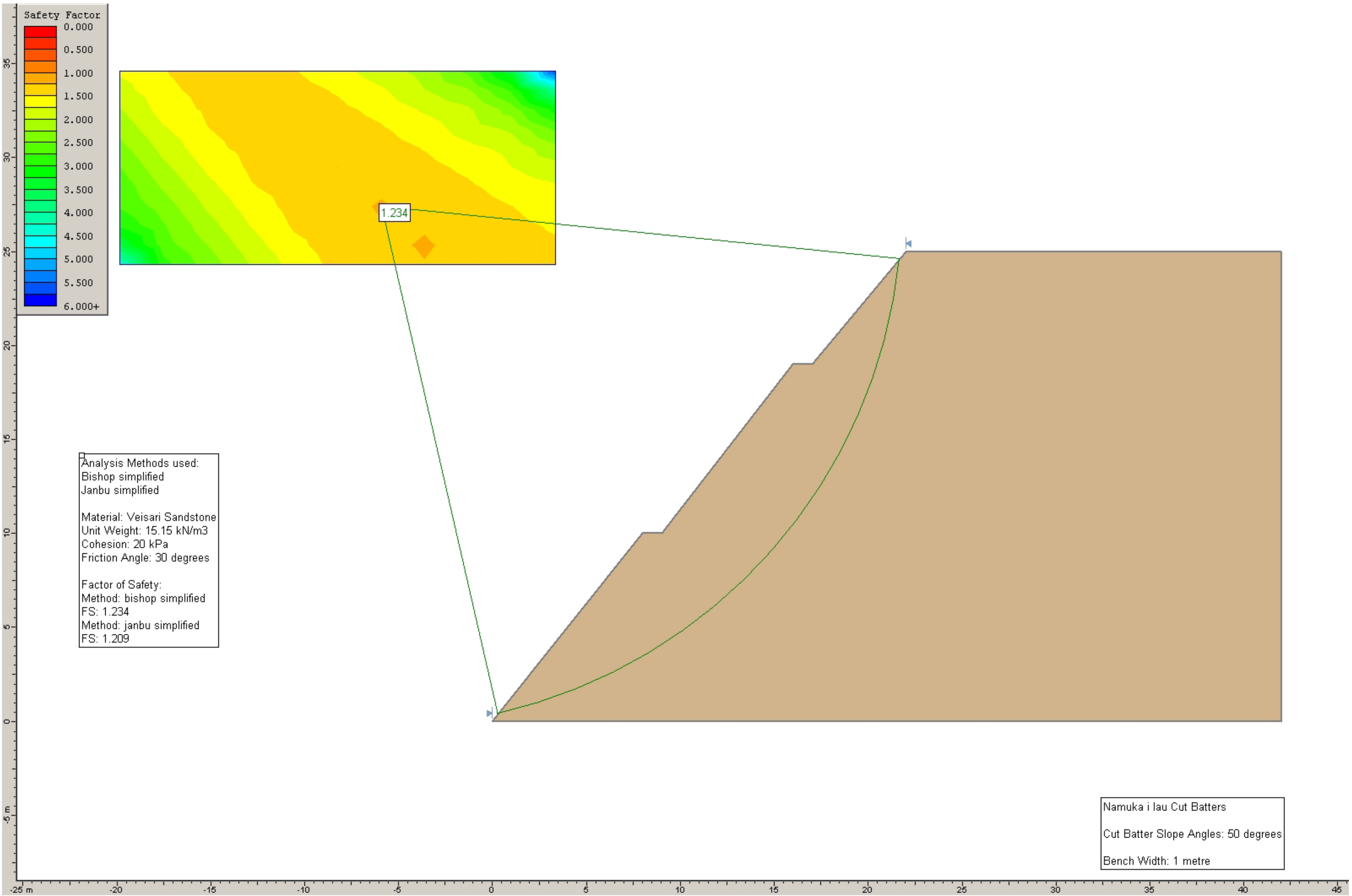
6.000+
1.091
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 20 kPa
Friction Angle: 25 degrees
Factor of Safety:
Method: bishop simplified
FS: 1.091
Method: janbu simplified
FS: 1.089
Namuka i lau Cut Batters
Cut Batter Slope Angles: 50 degrees
Bench Width: 1 metre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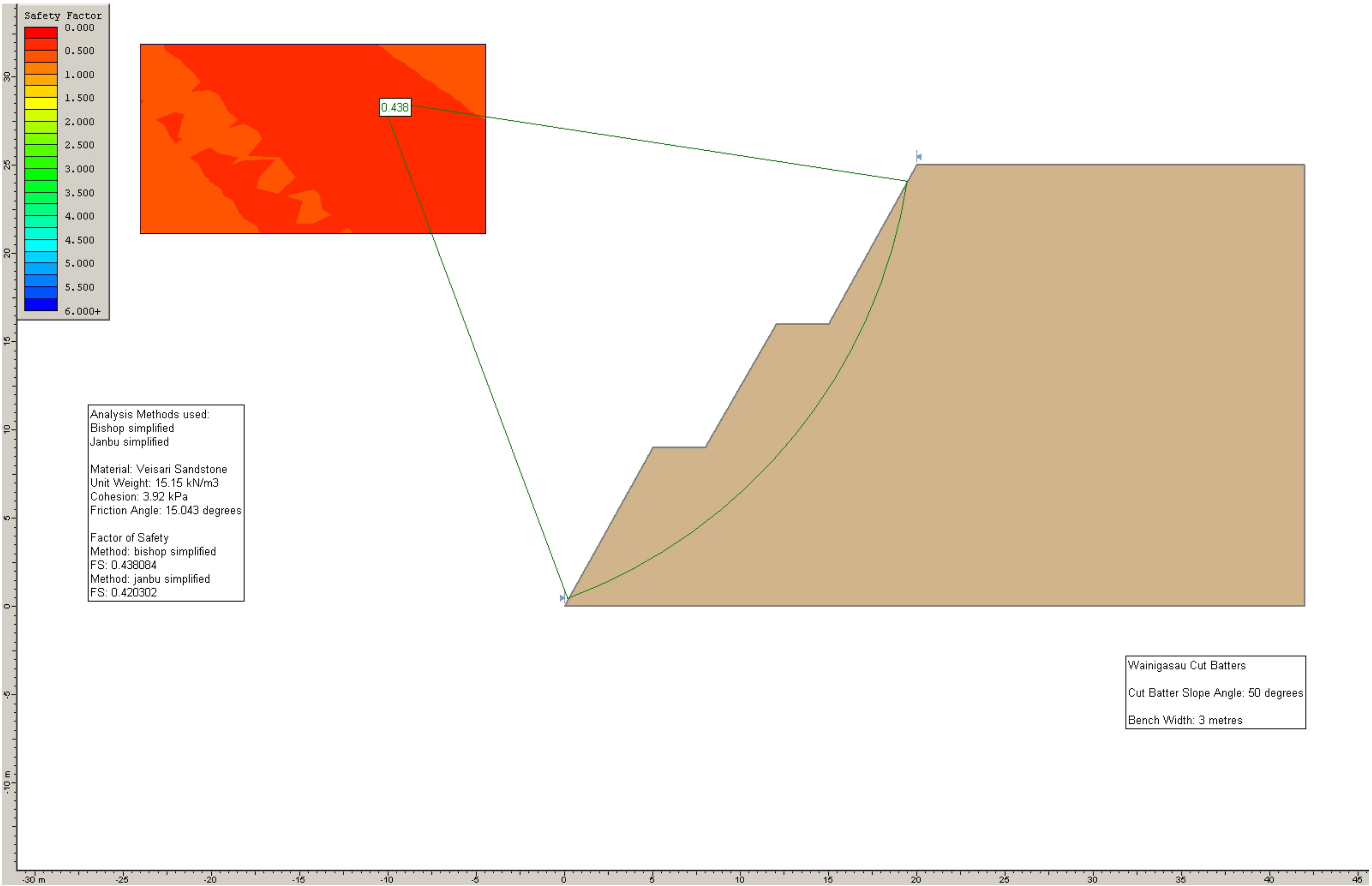
1.234
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 20 kPa
Friction Angle: 30 degrees
Factor of Safety:
Method: bishop simplified
FS: 1.234
Method: janbu simplified
FS: 1.209
Namuka i lau Cut Batters
Cut Batter Slope Angles: 50 degrees
Bench Width: 1 metre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Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 3.92 kPa
Friction Angle: 15.043 degrees
Factor of Safety
Method: bishop simplified
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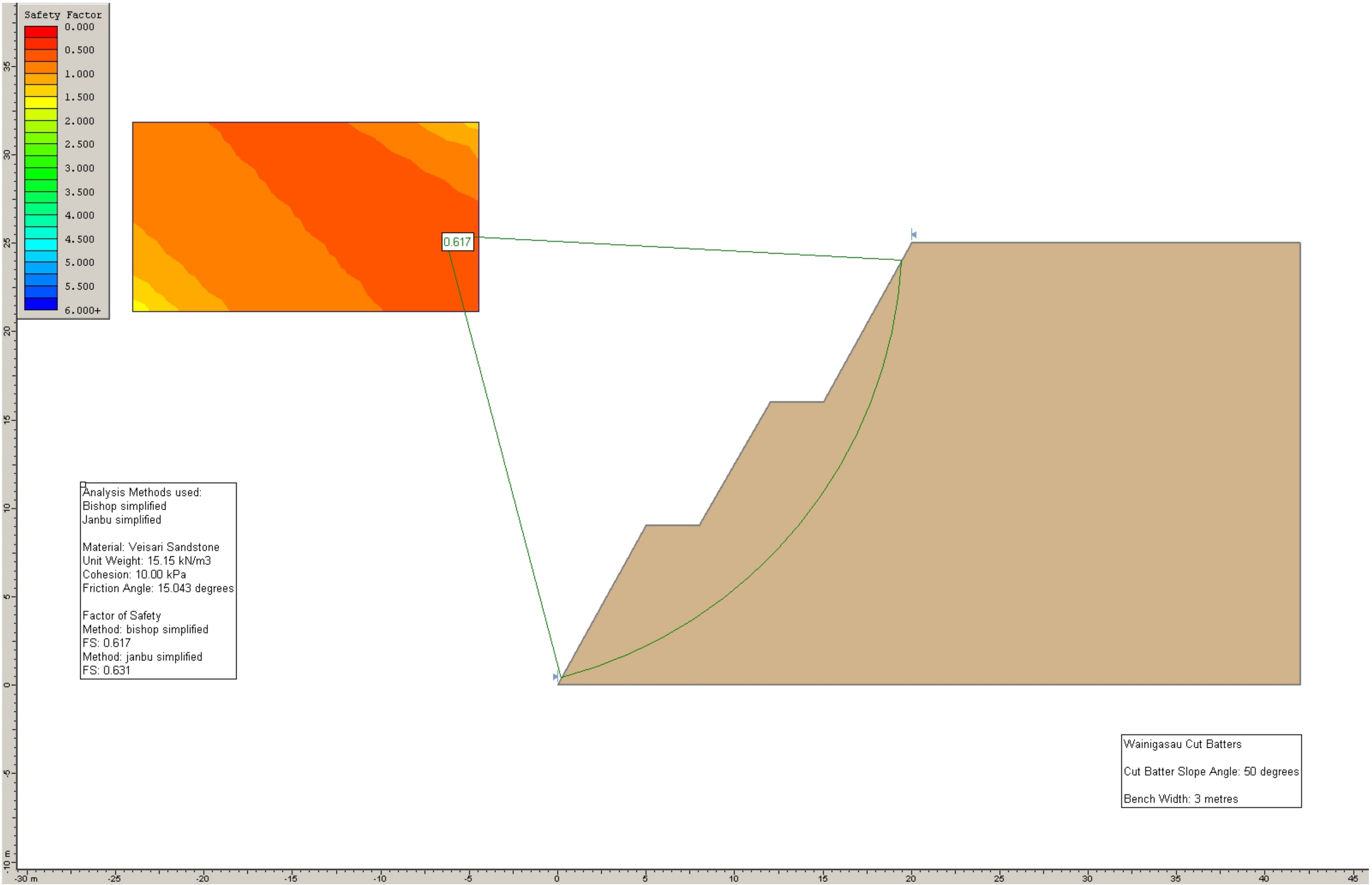
Method: janbu simplified
FS: 0.420302
Wainigasau Cut Batters
Cut Batter Slope Angle: 50 degrees
Bench Width: 3 metres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Analysis Methods used:
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Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 10.00 kPa
Friction Angle: 15.043 degrees
Factor of Safety
Method: bishop simplified
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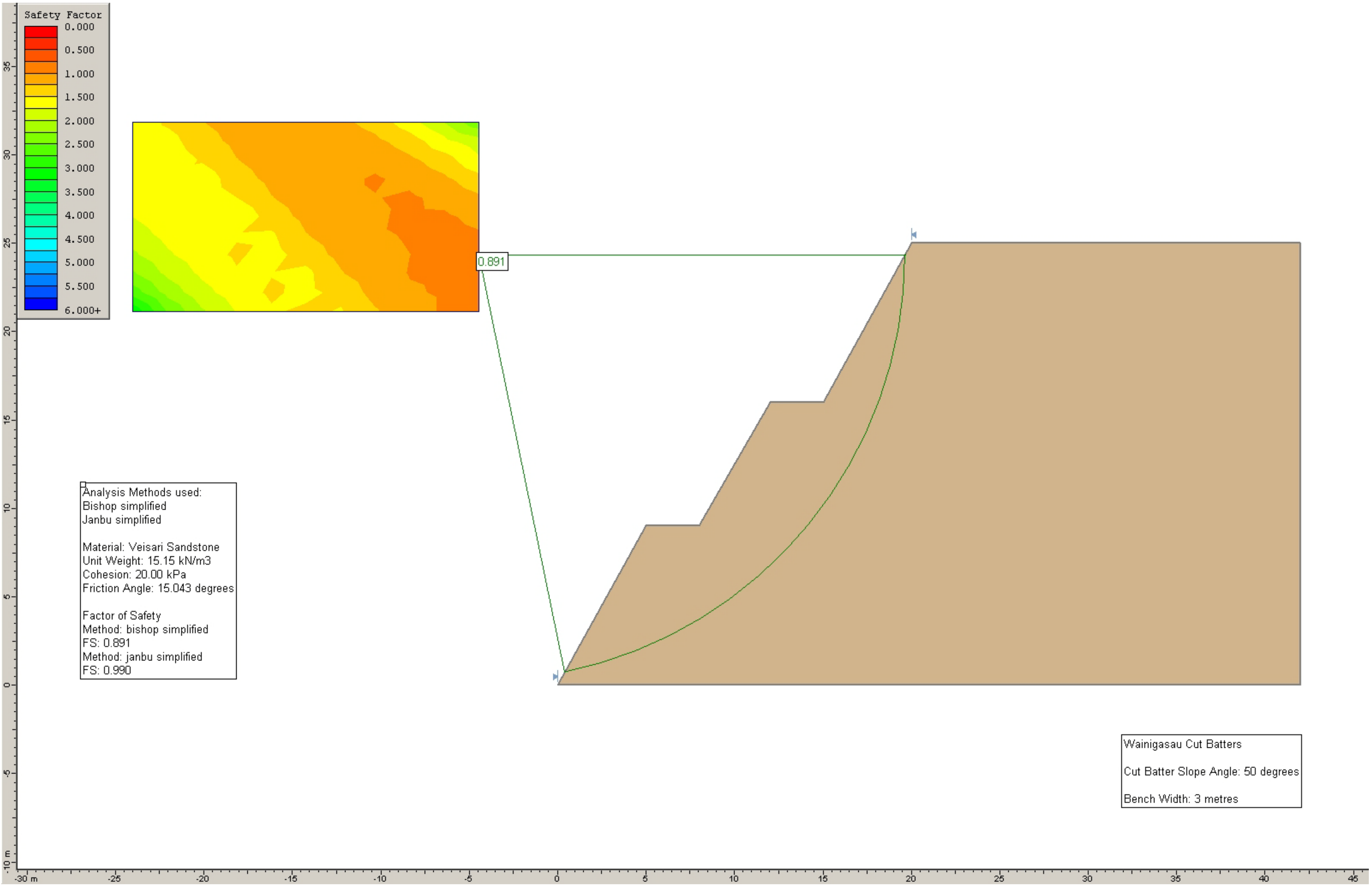
Method: janbu simplified
FS: 0.631
Wainigasau Cut Batters
Cut Batter Slope Angle: 50 degrees
Bench Width: 3 metres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5.500
6.000+
0.891
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 20.00 kPa
Friction Angle: 15.043 degrees
Factor of Safety
Method: bishop simplified
FS: 0.891
Method: janbu simplified
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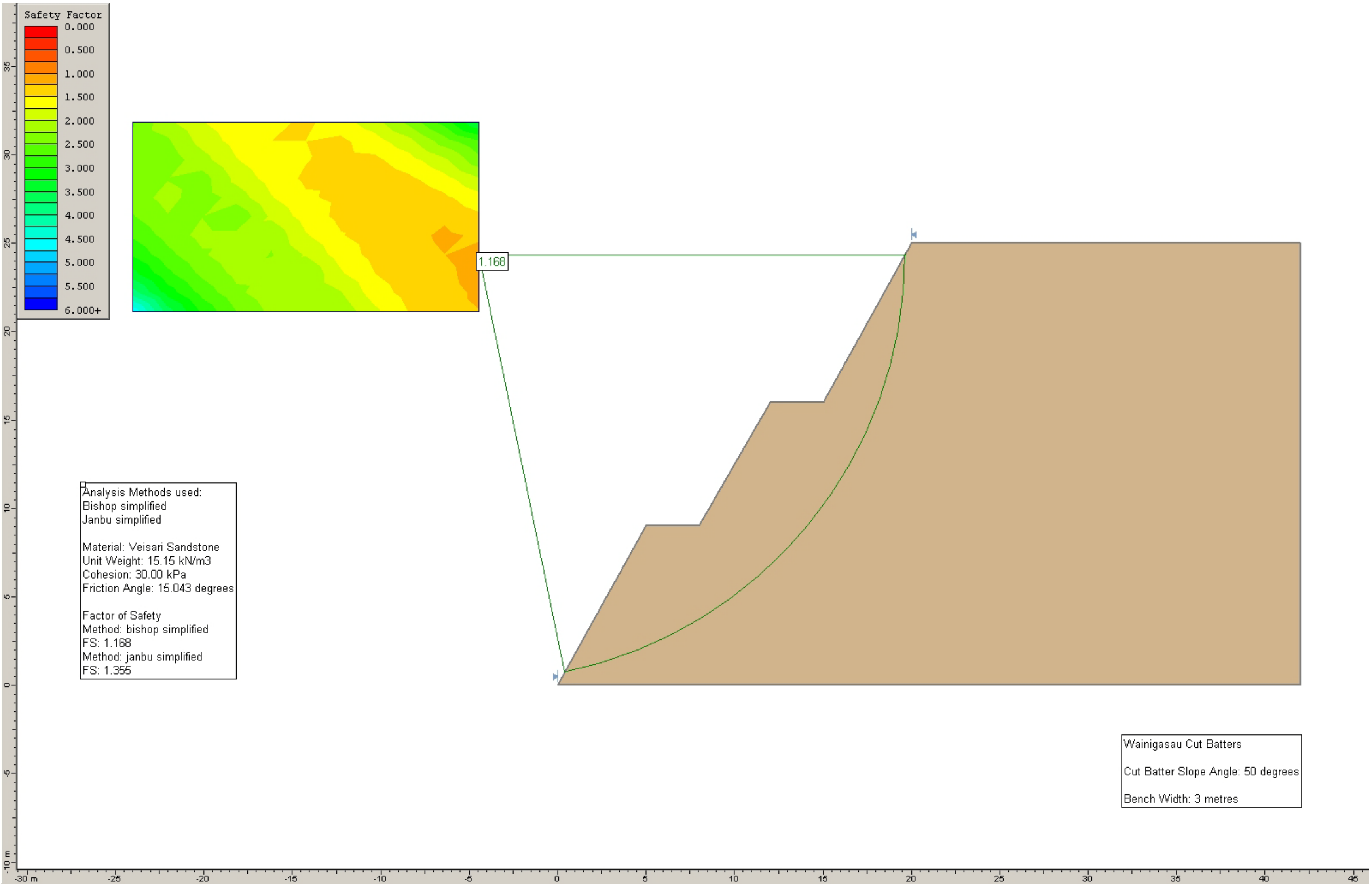
Wainigasau Cut Batters
Cut Batter Slope Angle: 50 degrees
Bench Width: 3 metres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5.500
6.000+
1.168
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 30.00 kPa
Friction Angle: 15.043 degrees
Factor of Safety
Method: bishop simplified
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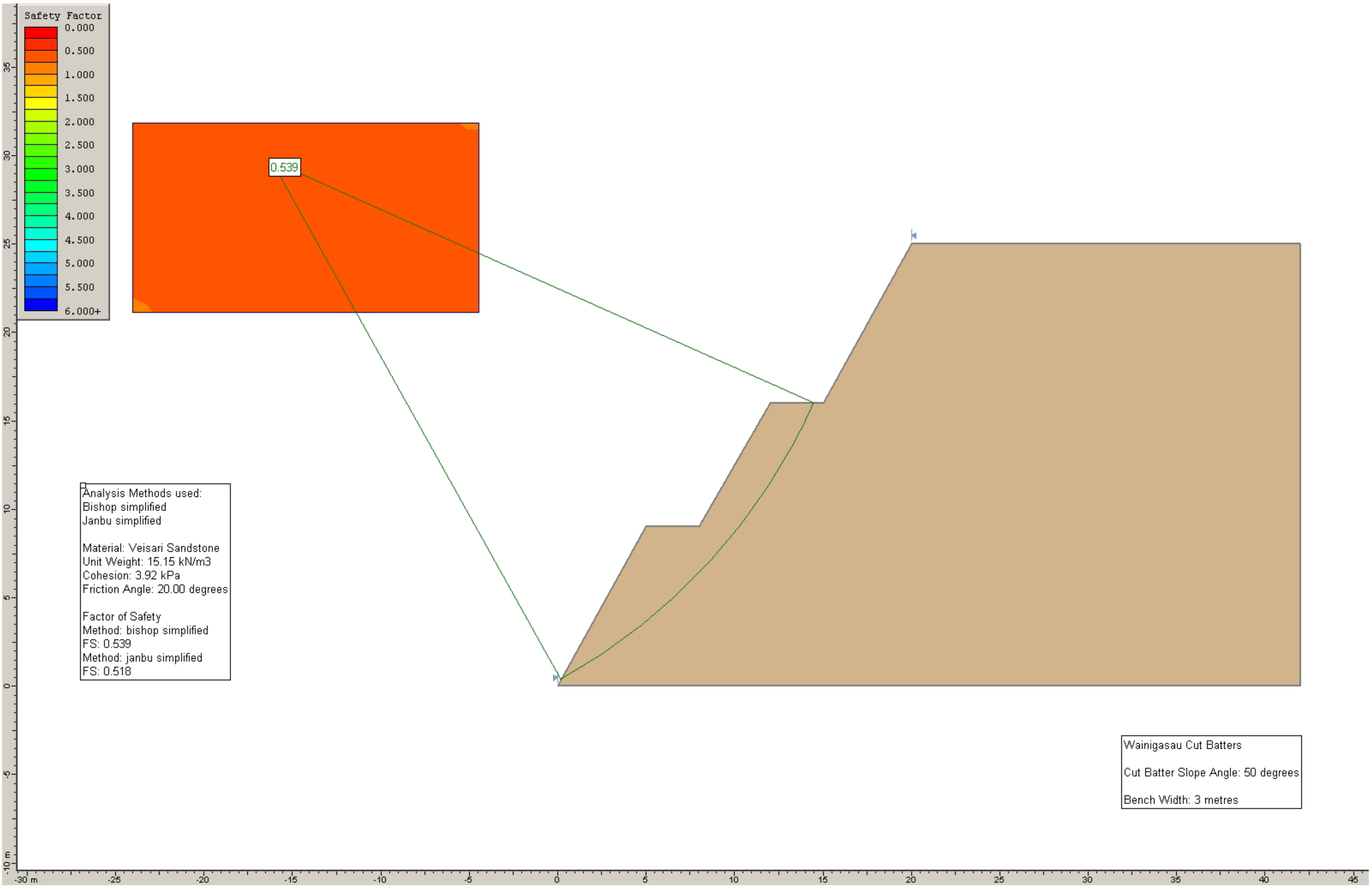
Method: janbu simplified
FS: 1.355
Wainigasau Cut Batters
Cut Batter Slope Angle: 50 degrees
Bench Width: 3 metres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5.500
6.000+
0.539
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 3.92 kPa
Friction Angle: 20.00 degrees
Factor of Safety
Method: bishop simplified
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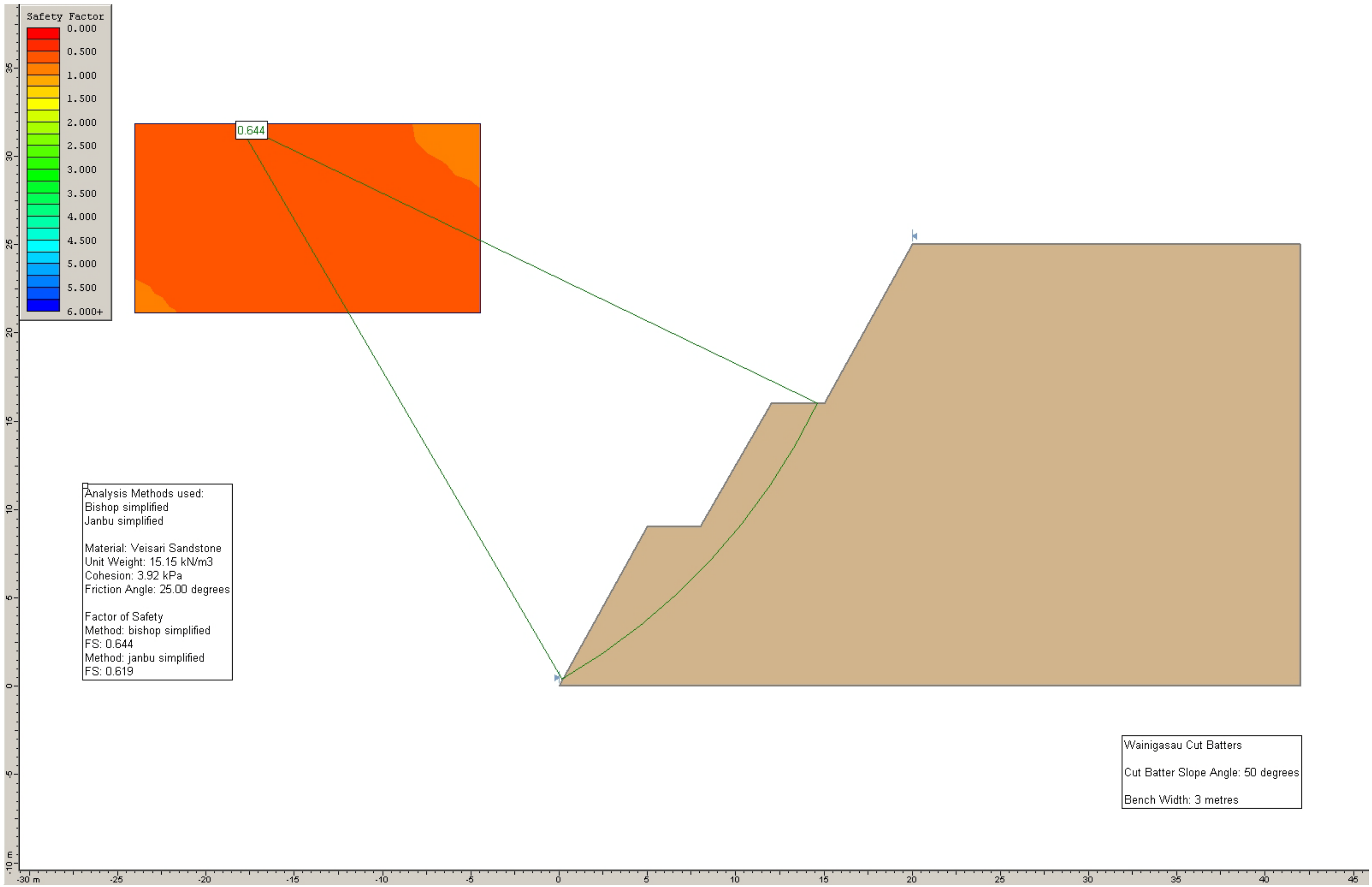
Method: janbu simplified
FS: 0.518
Wainigasau Cut Batters
Cut Batter Slope Angle: 50 degrees
Bench Width: 3 metres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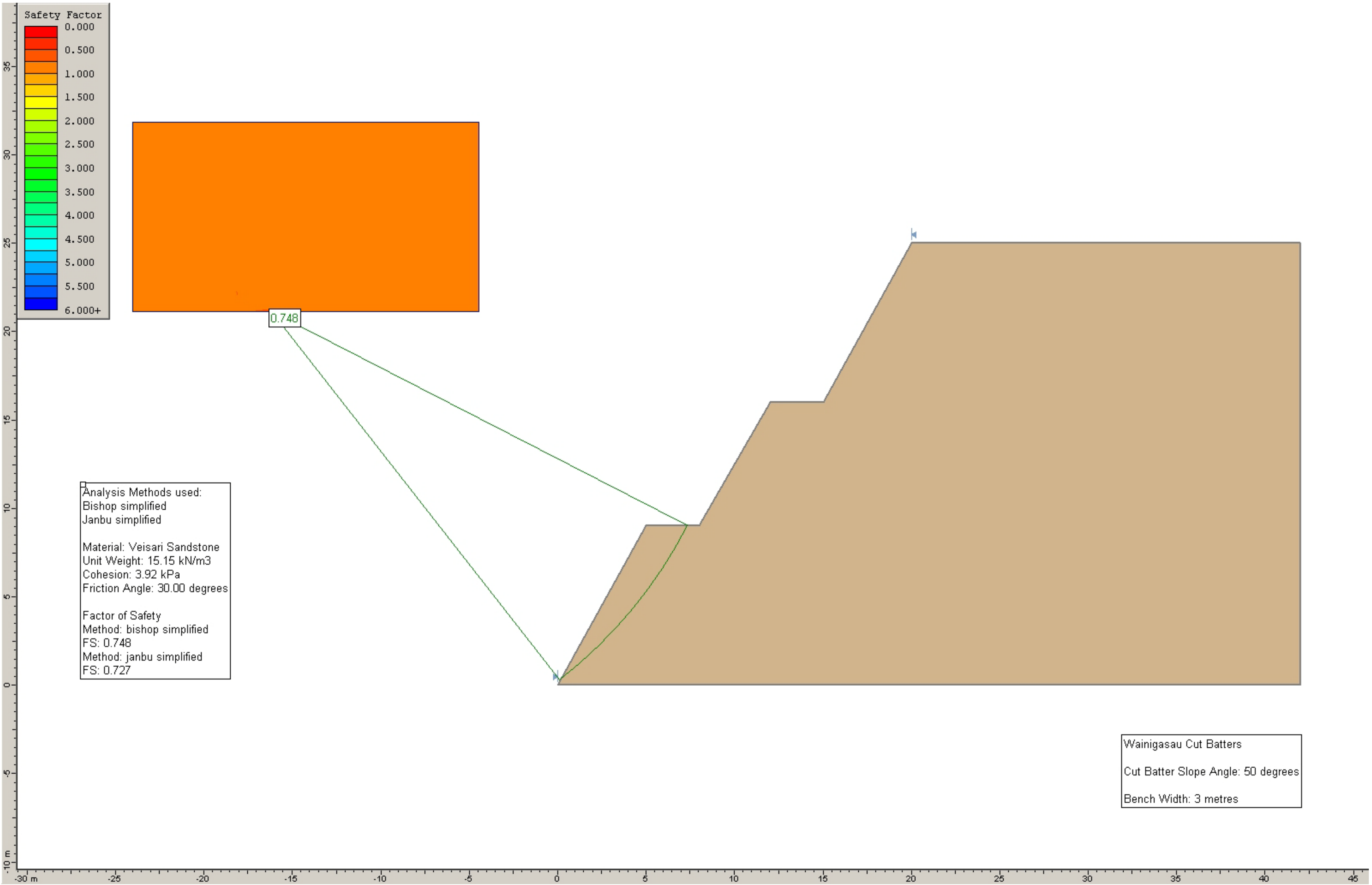
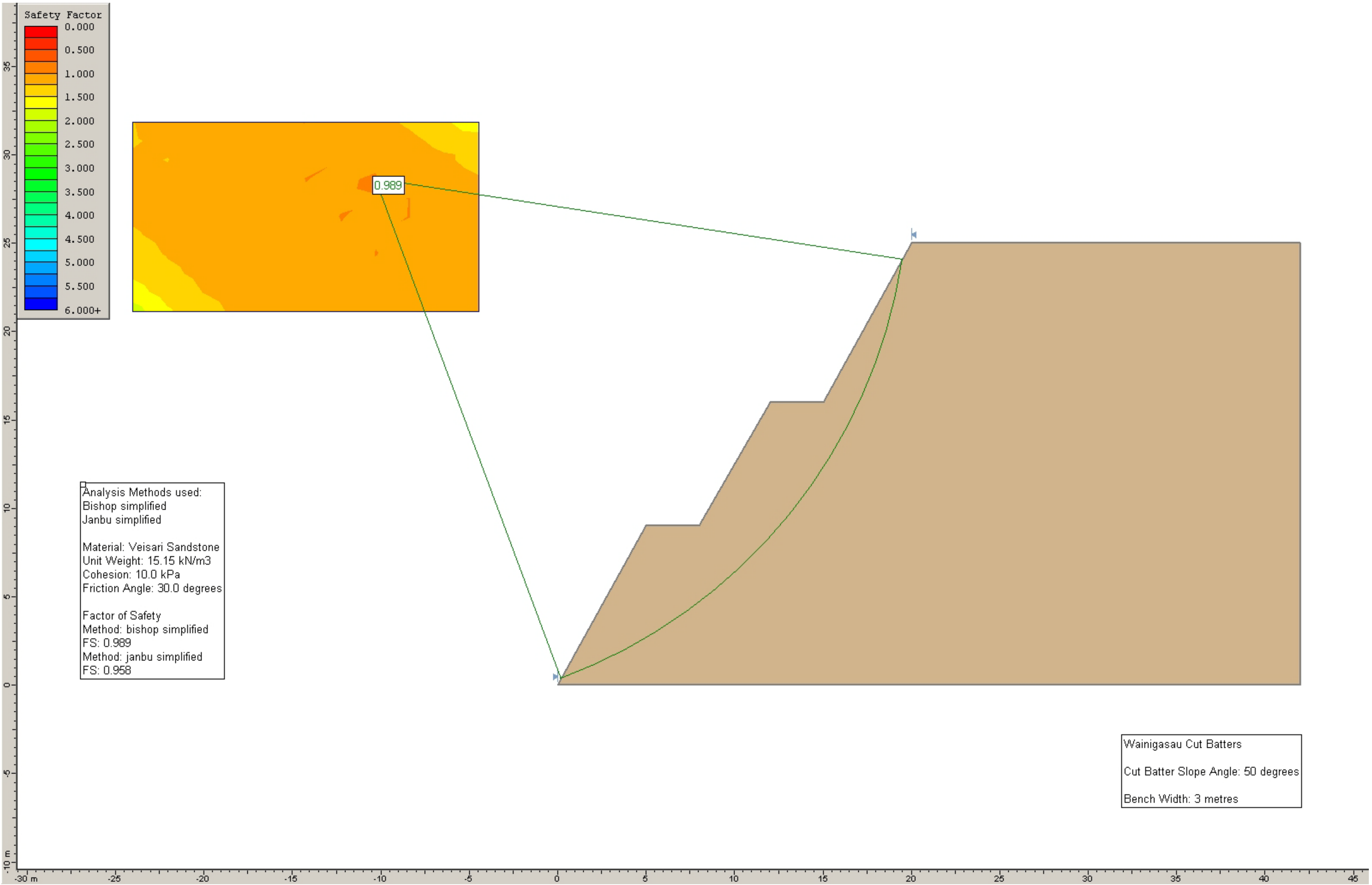
5.500
6.000+
0.644
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 3.92 kPa
Friction Angle: 25.00 degrees
Factor of Safety
Method: bishop simplified
FS: 0.644
Method: janbu simplified
FS: 0.619
Wainigasau Cut Batters
Cut Batter Slope Angle: 50 degrees
Bench Width: 3 metres


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.748
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 3.92 kPa
Friction Angle: 30.00 degrees
Factor of Safety
Method: bishop simplified
FS: 0.748
Method: janbu simplified
FS: 0.727
Wainigasau Cut Batters
Cut Batter Slope Angle: 50 degrees
Bench Width: 3 metres


Safety Factor
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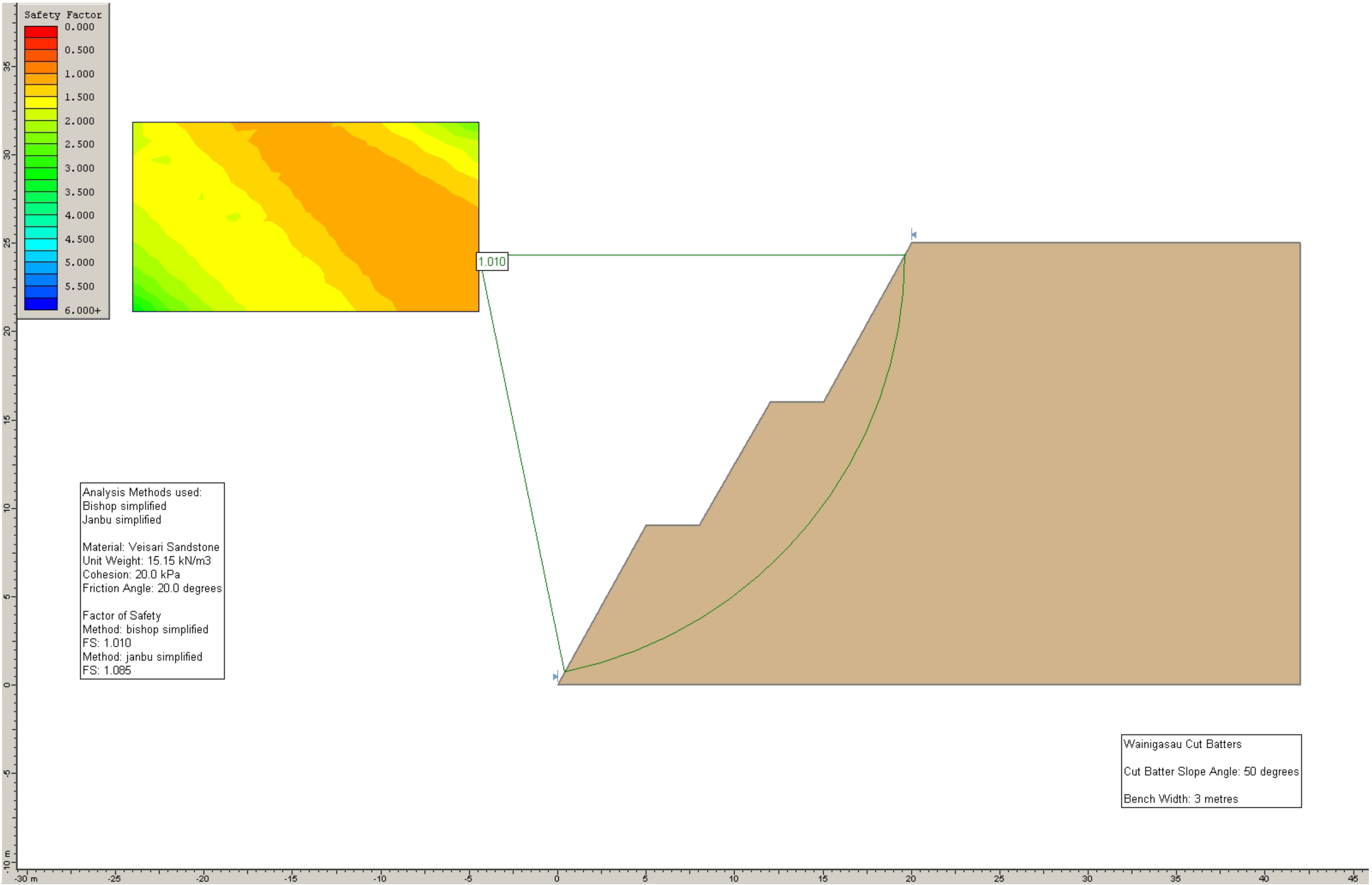
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.989
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 10.0 kPa
Friction Angle: 30.0 degrees
Factor of Safety
Method: bishop simplified
FS: 0.989
Method: janbu simplified
FS: 0.958
Wainigasau Cut Batters
Cut Batter Slope Angle: 50 degrees
Bench Width: 3 metres


Safety Factor
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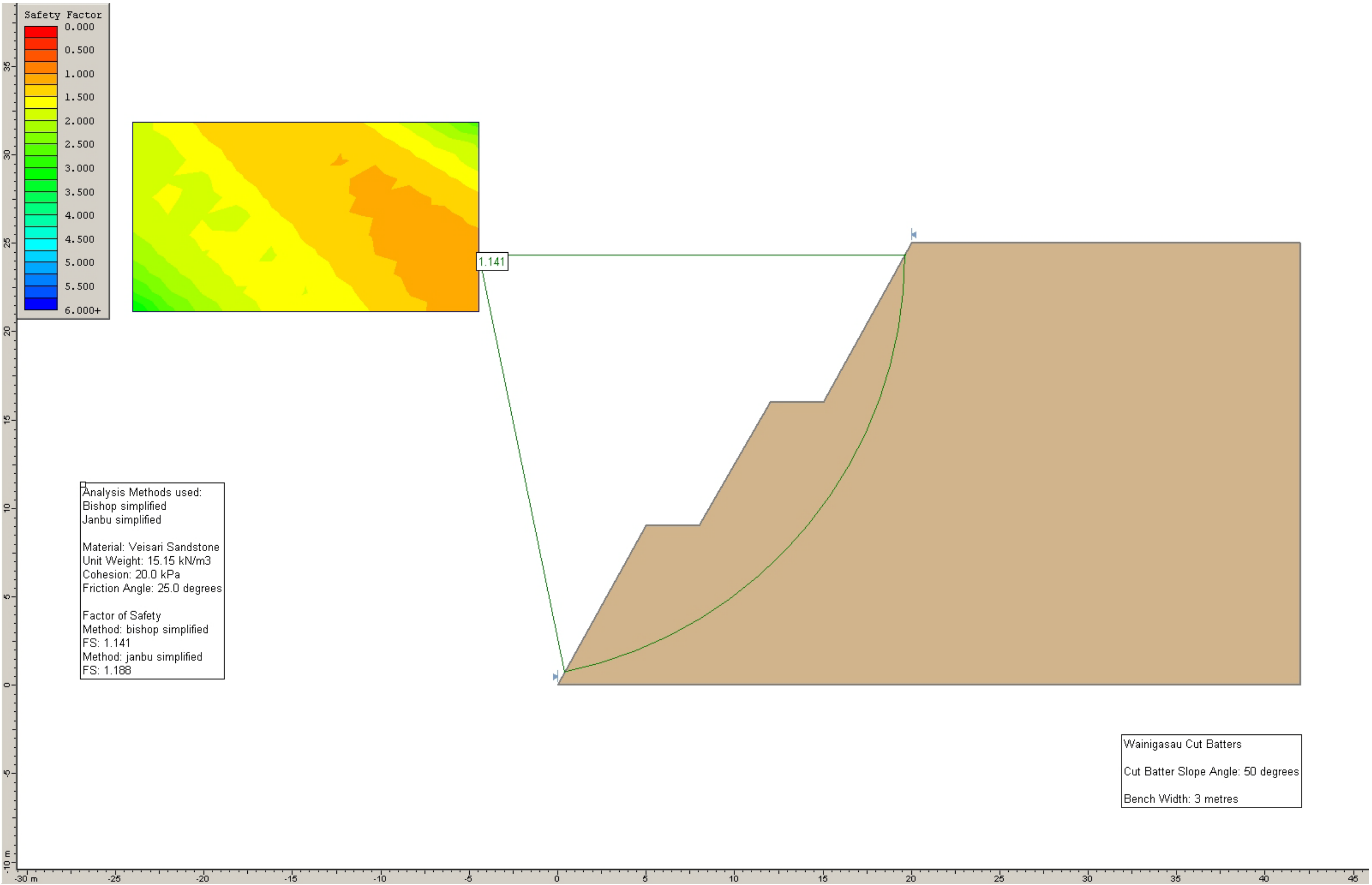
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.010
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 20.0 kPa
Friction Angle: 20.0 degrees
Factor of Safety
Method: bishop simplified
FS: 1.010
Method: janbu simplified
FS: 1.085
Wainigasau Cut Batters
Cut Batter Slope Angle: 50 degrees
Bench Width: 3 metres


Safety Factor
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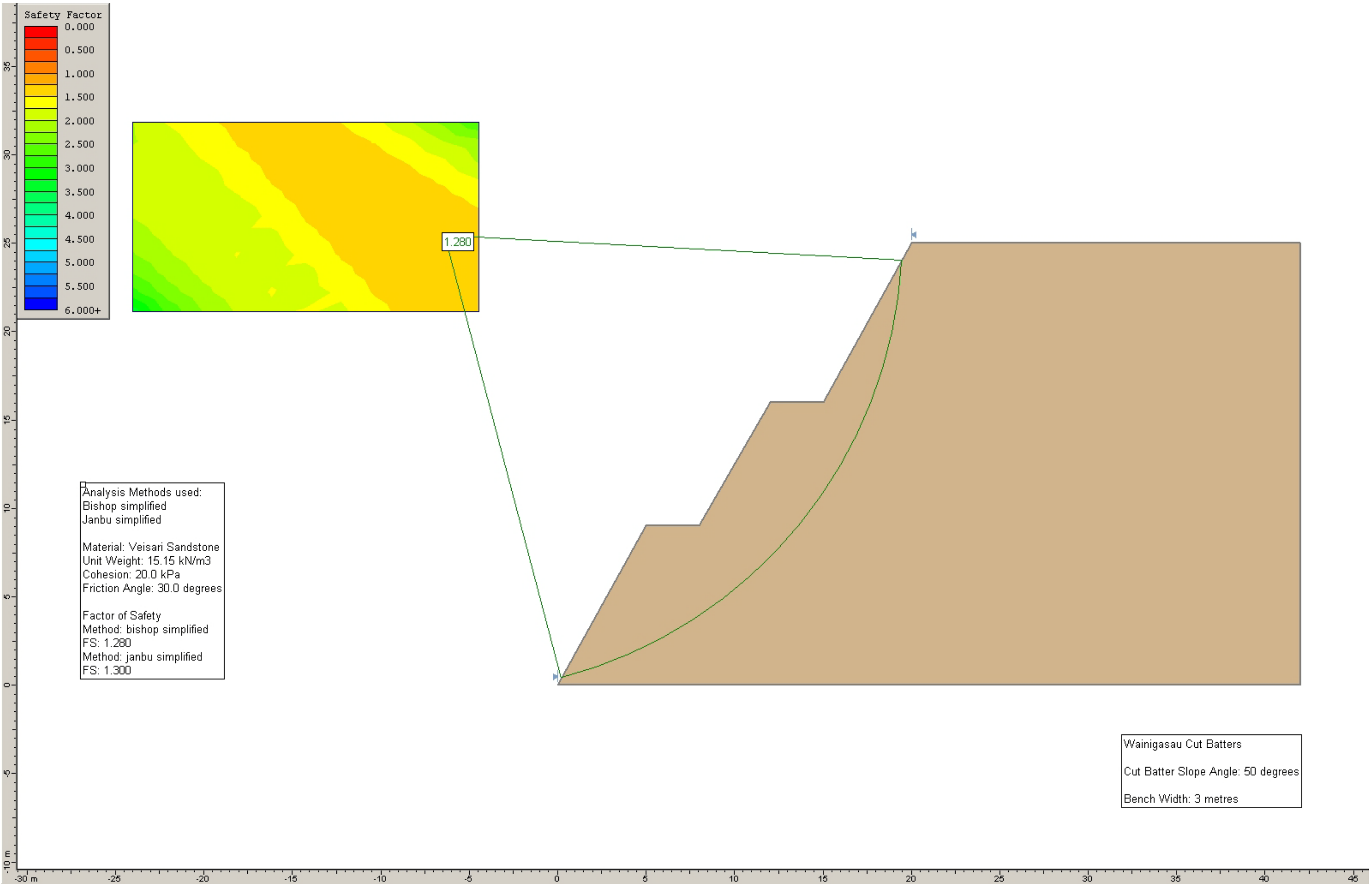
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.141
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 20.0 kPa
Friction Angle: 25.0 degrees
Factor of Safety
Method: bishop simplified
FS: 1.141
Method: janbu simplified
FS: 1.188
Wainigasau Cut Batters
Cut Batter Slope Angle: 50 degrees
Bench Width: 3 metres


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.280
Analysis Methods used:
Bishop simplified
Janbu simplified
Material: Veisari Sandstone
Unit Weight: 15.15 kN/m3
Cohesion: 20.0 kPa
Friction Angle: 30.0 degrees
Factor of Safety
Method: bishop simplified
FS: 1.280
Method: janbu simplified
FS: 1.300
Wainigasau Cut Batters
Cut Batter Slope Angle: 50 degrees
Bench Width: 3 metres